ACI News Letter

Vol. 21 No. 4 JOURNAL of the AMERICAN CONCRETE INSTITUTE December 1949

MEMORANDUM *from* ACI

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To ACI MEMBERS

SUBJECT ..... 46th ANNUAL CONVENTION

PROGRAM TO INCLUDE --

INSPECTION
BUILDING CODE PROBLEMS
PRECAST STRUCTURES
PRESTRESSING
LONG-SPAN CONSTRUCTION
RESEARCH
ADMIXTURES
STRUCTURAL DESIGN OF PAVEMENTS
R/C DESIGN
DEFORMED BARS
FOOTINGS
CORROSION OF REINFORCEMENT

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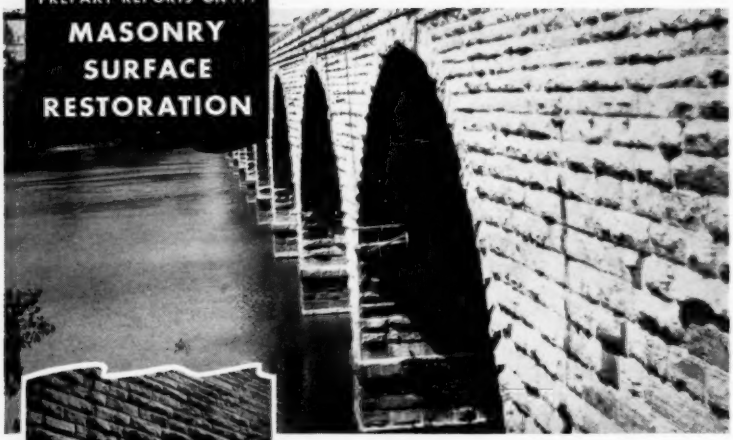
Technical Activities Committee

PREPAKT REPORTS ON . . .

MASONRY SURFACE RESTORATION



The world-famous Great Northern Railway Stone Arch Bridge in Minneapolis. General view of bridge and Prepakt restoration of damaged masonry.

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New England Regional Meeting a Success

New England ACI Members and their friends, as well as visiting Board and TAC members, will long remember the Regional Meeting in Boston, November 10 and 11, as one of cordial sociability and high technical interest. Registration reached a total of 194 and there were probably 30 or more who for one reason or another did not register.

General Session

The meeting opened 11 o'clock, Thursday, November 10, with Director Paul W. Norton of Boston presiding. Distinguished guests who conveyed the greetings of the state, city, educational and technical groups of the region were: Joseph Dever, representing his brother, Governor Dever of the Commonwealth of Massachusetts; Thomas Flynn, Deputy Building Commissioner, representing Mayor Curley of the City of Boston; James R. Killian, President of Massachusetts Institute of Technology, representing the engineering schools of the area; and Carroll Farwell, President of the Engineering Societies of New England.

In responding to the cordial welcomes ACI President Gilkey recalled that Boston has played a large part in the growth of ACI. Notable members include the late Sanford E. Thompson, class of '88 M.I.T., former ACI vice-president, who was elected to Honorary membership in 1932 and who had been a member of ACI since 1906; his was and will continue to be a great name in concrete. The late Leonard C. Wason, second president of the Institute and founder of two of ACI's award medals was of M.I.T.'s class of '91. He joined ACI in 1908 and was made an Honorary member in 1932. ACI patriarchs are well concentrated and represented in the Boston region.

Continued on p. 4

A Study in Concrete

The main address of the opening session was by Prof. C. H. Scholer of Kansas State College. In his whimsical way, he speculated on how cement and concrete first were used by prehistoric man; how a cement may have been discovered by accident when some fellow building a fire calcined the surrounding rocks and then discovered that by mixing the resulting powder with water he could reproduce a hard rock-like material. Professor Scholer then sketched the history of cements and concrete from ancient uses in Egypt and in our own southwestern primitive structures through the work of Smeaton on Eddystone Lighthouse and Joseph Aspdin's patent of the first portland cement; through its development in Europe and later in the United States; the use of natural cements from mills in the Lehigh Valley with considerable success in the mid-1800's; to the rapid development of the portland cement industry in the U. S. beginning about 1890. He then talked of specifications and other things, saying that perhaps concrete is not as good as it should be, but it is a commendable material, the best available for its uses, and everything considered, it has served well. He whimsically cited inconsistencies in the performance of concrete which are contrary to the things we *think* we *know* about it. He effectively conveyed to the audience the fact that we still have a lot to learn.

Luncheon

At an informal noon luncheon a group of 100 or more, including students from the nearby engineering schools, heard Director Henry L. Kennedy introduce the officers and directors of ACI. President Gilkey spoke briefly about the Institute and its activities.

First Technical Session

Thursday afternoon Prof. F. N. Weaver of the Civil Engineering Department, Tufts College, presided at a technical session of excellent papers and discussion.

Pozzolanic materials

Prof. Raymond E. Davis, ACI Past President, Director, Engineering Materials Laboratory, University of California, Berkeley, spoke on "Pozzolanic Materials and Their Use in Concrete." He described the many beneficial characteristics which in some instances are imparted to concrete by the use of these pozzolans as replacements for varying percentages of the cement. Interest in the subject was reflected in the discussion which followed, O. G. Julian, Wm. D. O'Connell and R. B. Young contributing.

Long-span arch hangar

John E. Allen, Chief, Engineering Division, U. S. Engineer Office, Boston District, presented an excellent and well illustrated construction paper entitled "Construction of Long-Span Concrete Arch Hangar at Limestone Air Force Base."

Concreting large dams

A paper on "Concreting on the Ottawa Valley Project of the Hydro-Electric Power Commission of Ontario" was presented by A. L. Malcolm, Assistant Engineer, Hydro-Electric Power Commission of Ontario, Toronto, Ontario, Canada. Mr. Malcolm spoke largely on construction plant layout and use of equipment. Of notable interest was the wide use of surplus Bailey bridge sections. R. B. Young, Assistant Director of Research of the Hydro-Electric Power Commission conducted the discussion and described the use of high lifts—usually 50 ft—in placing concrete. This practice is not generally used in the U. S. but Mr. Young said the Hydro-Electric Power Commission has found no cracks in its dams attributed to this procedure so they are continuing to use the method as they have been for almost 20 years. In reply to questions by O. G. Julian, Mr. Young said that they have an extensive investigation in hand trying to determine maximum form pressures. He also said that on their jobs, the laboratory becomes a sort of unofficial club so that the construc-

tion force from the top down will find a personal interest in what makes good concrete.

Power plant construction

W. S. Colby, Structural Engineer, Stone and Webster Engineering Corp., Boston, described the construction problems met and solved in the "Design and Construction of a Circulating Water Intake for Venice No. 2 Power Plant, Union Electric Power Co." at Venice, Ill., on the Mississippi River opposite St. Louis, Mo. Construction problems were unique in that the plant is on the land side of the levee where the water level in the river varies from a gage reading of −5.0 to +47.6 ft. The fluctuation of the river level had considerable influence on the arrangement of the circulating water system. George Flay added a note of interest that the 75-ft high concrete pump well structure was placed in four lifts.

Dinner

A pleasant interlude between the technical sessions was the dinner on Thursday evening at which Albert Haertlein, Professor of Civil Engineering, Harvard University, presided as chairman.

The speaker of the evening was Donald Cope McKay, Professor of History, Harvard University, who titled his talk "France Turns the Corner."

He explained that that title represented the impressions he had received from a visit of several weeks to France this past summer during which he talked to a good many American and French officials. He added that, unfortunately, the situation has now been modified in some respects so that the turn of the corner is somewhat less clear.

"On the favorable side," said Professor McKay, "is to be noted particularly the fact that France has made a remarkable recovery in the field of transportation and production generally. The franc has become a relatively hard currency. The Queuille government managed to last for more than a year, which was a sign of some returning political stability.

"At the same time the strength of the Communists has declined somewhat and with it the strength of the Gaullists, so that the parties of the two extremes have been in a less favorable position to harass the parties of the Third Force, which were of course the supporters of Queuille. It goes without saying that the Marshall Plan has made a very large contribution to increased production.

"In the 'unfavorable' column," he continued, "is the fact that inflation, although it was brought under control as of the end of 1948, with wholesale and retail prices dropping during the first half of this year, has now resumed an upward trend and the cost of living is again just about at its peak. Exports . . . have moved up slowly to the non-dollar areas and more slowly to the dollar areas. Meanwhile, the war in Indo-China makes a heavy drain on the budget and is a distinctly unsettling element.

"The Communists and De Gaulle are not immediate sources of weakness so much as important potential sources of weakness," he concluded. "If the economic situation worsens, or the present governmental set-up proves to be not viable, or Moscow decides once more to move forward in France, then both the Communists and the Gaullists could once again become important elements on the 'unfavorable' side."

Second Technical Session

The presentation of technical papers continued at a session on Friday morning with W. C. Voss, Professor of Building Construction, M.I.T., as chairman.

Pavement concrete

ACI Vice-President Frank H. Jackson, Principal Engineer of Tests, Bureau of Public Roads, Washington, presented "Essential Properties of Pavement Concrete," a concise review of performance requirements of concrete pavements from the standpoint primarily of the quality

of the concrete as a material. He pointed out that the lack of durability of much of our present-day concrete may be due to the methods of construction which we employ. He believes that a significant improvement in pavement durability could be effected by using a scientifically proportioned mix of dry consistency well compacted by vibration or tamping and with close control over aggregate gradation in place of the oversanded plastic mixtures which are now used. He believes that air entrainment is not necessarily the final answer to the problem of surface deterioration but that we should seriously consider overhauling our entire construction practice as well as our present method of controlling the uniformity of aggregate gradation. Brief floor discussion included questions from W. F. Swiger and W. H. Gumpertz.

Precast bridges

Going back to engineering work in New England, Robert C. Hanckel, Consulting Engineer, Boston, described, "Precast Units for Short-Span Bridges," in the city of Lowell, Massachusetts. Precast units were used to replace bridges where minimum traffic interruption was necessary. Four bridges of two types were described. In one type precast units made up the complete structure except for grouting joints and applying black top wearing surface and rails. In the other type precast units made up only part of the completed structure—the deck being poured in place to integrate the precast units into as nearly a monolithic whole as possible. Mr. Hanckel's paper provoked questions from the floor by H. G. Protze, W. H. Gumpertz and A. R. Barnes.

Armistice Day observance

A few minutes before 11:00, the technical program was interrupted for appropriate remarks in observance of Armistice Day by Gen. James F. McManmon, retiring Commissioner of Airports.

Precast building construction

Thomas F. Gilbane, President, Gilbane Building Co., Providence, R. I., presented "Precast Concrete Panel-Type Multi-Story Construction," describing in considerable detail the work his firm has done in speedy precast construction on jobs at the University of Connecticut and the Manchester Housing Project. Thomas T. Amirian remarked in discussion that he had gone through the buildings at Storrs, Conn., and had been much impressed.

Research

Hubert Woods, Director of Research, Portland Cement Association, Chicago, in "Research in Concrete Durability," dealt with the effects of freezing and thawing and the prevention of damage therefrom. His remarks were based on information garnered from field observations on both full scale and semi-full scale structures exposed to the elements, most of them constructed as a part of the "Long-Time Study of Cement Performance in Concrete" conceived by P. H. Bates. In the short discussion period F. H. Jackson, M. A. Swayze and Chairman Voss added questions or comments on Mr. Woods' subject.

Inspection Trips

As a fitting conclusion to the meeting, two inspection trips were provided. The shorter one to the concrete laboratories of Northeastern University provided a short trip for those who had to leave early in the afternoon. The longer trip took a full bus load of about 50 from the Kenmore Hotel through the Sumner Tunnel to the new East Boston Skyway and Rapid Transit construction where engineers on the job described the traffic interchange project which will link a number of Boston's principal arteries with the airport and routes leading north, south and west. The group was then taken to the upper deck of the new Mystic River Bridge to the toll house plaza where the resident engineer described the outstanding features of this two-mile long double-deck bridge project.

The group then proceeded to M.I.T. in Cambridge where it inspected the concrete laboratories of the civil engineering department and of the building materials department. Prof. R. J. Hansen described work of the civil engineering laboratory; Prof. A. J. Murray, the work in the building materials laboratory. Doughnuts and coffee were served and the group returned to the Hotel Kenmore.

On Thursday afternoon the ladies were treated to a sightseeing tour of historic Boston concluding with tea at T-Wharf.

Committee congratulated

At the close of the Thursday morning session, ACI Senior Vice-President Jackson expressed the appreciation of the officers and members of the Institute for the tremendous amount of work done by the local committee which made the meeting such a great success. "As far as I am concerned," said Mr. Jackson, "it has been a success all along the line and I think you will agree with me that the papers, not mentioning my own, have been of very high order, and I think the program committee should be highly complimented on its efforts to secure a worthwhile program.

"All other functions have been very well executed and we will go away from Boston highly pleased with the success of this meeting. I hope," he concluded, "we will have an opportunity to come back shortly and participate in your wonderful hospitality."

Papers to be published

All papers presented are being considered for JOURNAL publication and with the cooperation of the authors will be published as soon as possible. Mr. Woods' paper is the exception, since it is based largely on material already published or to be published in connection with the Long-Time Study.

Committee Personnel

General Committee

Henry L. Kennedy, Co-chairman
Paul W. Norton, Co-chairman
Miles N. Clair
L. C. Blake
Emil A. Gramstorff
Dean Peabody, Jr.

Program Committee

Miles N. Clair, Chairman
Albert Haertlein
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Publicity Committee

Dean Peabody, Jr.
Ray Welts

Inspection Trips and Ladies' Program

E. M. Brickett
F. A. McAdams

Business Management

Emil A. Gramstorff

ERRATA

The following corrections should be made in the paper "Bond of Concrete Reinforcing Bars" by Arthur P. Clark, ACI JOURNAL, Nov. 1949.

p. 168—Captions for figures should read:

Fig. 5 (top)—Bond-slip curves for $\frac{7}{8}$-in. top bars in beams, slip measured at free end

Fig. 6 (bottom)—Bond-slip curves for $\frac{7}{8}$-in. top bars in beams, slip measured at loaded end

1950 CONVENTION PROGRAM NEAR COMPLETION

The Technical Activities Committee at its meeting in Boston, November 9, approved items for the 1950 convention program bringing it almost to final arrangement. A few items which are assured remain to be approved in their final manuscript form.

As now set up, the program includes a session on inspection for which J. W. Kelly, Chairman of Committee 611, Inspection of Concrete, is serving as sponsor and chairman. The following titles are included:

"Inspection and Testing of Materials," "Inspection of Building Construction," "Inspection and Control of Concrete for Highway and Bridge Construction," "Inspection of Mass and Related Concrete Construction," "The Contractor's Viewpoint of Inspection," "Inspection of Ready-Mixed Concrete," and "A Summary of Inspection Practices."

There will be a session devoted to ACI Building Code studies and to problems common to Committees 208, Bond Stress; 318, Standard Building Code; 323, Prestressed Reinforced Concrete; 324, Precast Reinforced Concrete Structures; and 711, Precast Floor Systems for Houses. Titles pertaining to the Code are: "Deformed Bars and Allowable Stresses," "Footing Design" and "Recommended Design Specifications for Two-Way Floor Floor Slabs."

Problems common to the several committees include: "Placing of Moment Bars in Precast Joists," "Extent and Acceptability of Cracking in Thin-Cover Precast Concrete Framing Members," "The Patent Status of Prestressed Concrete" and "Corrosion of Reinforcement in Thin-Cover Precast Concrete Framing Members."

There will be a session devoted to reinforced concrete design problems including: "Long-Span Construction," "Prestressed Reinforced Concrete," "A Simple Mathematical Tool for the Solution of Difficult Design Problems," "Precast Concrete Construction in Europe" and "Precast Reinforced Concrete Structures."

A session on structural design of concrete pavements is being developed by L. W. Teller, Chairman of ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports. Titles include: "General Problems of Structural Design of Concrete Pavements," "Subgrades and Bases," "Slab Dimensions," "Structural Design of Joints" and "Steel Reinforcement."

A part-session on admixtures is being developed by W. T. Moran of the U. S. Bureau of Reclamation, Denver. Titles will include an introduction and "Uses of Admixtures to Increase Resistance to Freezing and Thawing," "Admixtures in Underwater Construction," "Admixtures in Correcting Grading Deficiencies" and "Admixtures to Overcome False Set."

A number of items are being considered to fill out a half-session remaining unfilled but have not been selected for announcement.

C. H. Scholer has been selected to preside at a panel discussion of questions on concrete problems which have been submitted by Members of the Institute in reply to a questionnaire sent out in the fall of 1948.

As in past years the annual open session of Committee 115, Research, under the leadership of S. J. Chamberlin, Chairman, and George W. Washa, secretary, will be of top interest.

As announced in previous issues of the JOURNAL, seven sessions will be presented in five periods with two periods being devoted to concurrent sessions.

The convention will use the excellent facilities of the Edgewater Beach Hotel in Chicago, February 20 to 22 inclusive. Although an adequate number of rooms has been assured ACI for out-of-town visitors, it would be a good idea to make your reservations early mentioning that they are being requested in connection with the ACI annual convention.

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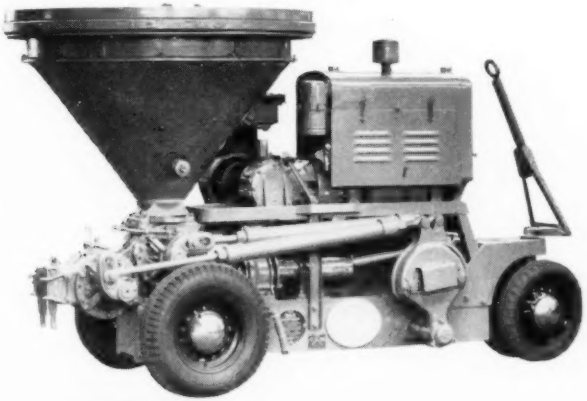

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Who's Who This Month

Nicolaas T. F. Stadtfeld

Mr. Stadtfeld, whose "Inspection and Testing of Materials" starts on p. 237, thinks that many engineers take themselves too seriously. That he does not wish to be so categorized is indicated by his biographical sketch which we quote:

Nicolaas T. F. Stadtfeld, "The Lone Ranger of the Kiln Room" often greeted as "Heigho Silver" in the meetings of Committee C-1, A.S.T.M., was born in 1883, without objecting, in Amsterdam, Holland. After graduating from Delft University as C.E. in 1905 he came to the United States in 1906 and at once plunged under the East River helping to build the Pennsylvania Railroad tunnels as instrumentman. In 1909 he went with the New York Central Railroad as assistant engineer realigning curves and designing structures, but found that sitting behind a drafting board all day brought him as close to death as did an electric locomotive sneaking up on him while he was surveying. So back into construction work in 1912 on grade crossing eliminations by the Long Island Railroad as chief inspector. Those were the days of high towers and soupy concrete, but the late L. V. Morris, then Chief Engineer, would have none of it, and concrete was placed at what now would be considered a 3- to 4-in. slump. It is still there in excellent condition, only the thin outer mortar coat having disappeared with the years. After some years spent on valuation of structures he became bridge inspector, visiting mills and fabricating shops and inspecting existing structures.

In 1923 he went with the City of New York in charge of materials inspection for the Staten Island Tunnel which was abandoned after two shafts were built. Transferred to the Board of Transportation which built subways, he had charge of the physical laboratory and checked up on central mixing plants and field concrete. Proportioning was changed from volumetric to weight measurements; water was closely controlled and compressive strength of concrete cylinders at 28 days went up from 1500 to 4000 psi. When the immoderate recession of the thirties caught up with him he went to work for the U. S. Army Engineers in 1934 and directed the laboratory at Fort Peck, Mont., again keeping in close touch with field work. Research on portland cement was a big item under the late Thaddeus Merriman and the laboratory covered such items as health service for the town, sanitary survey of part of the Missouri River, pressure of concrete against slanting forms, cement, aggregate and metals investigations, as well as many thousands of soil tests, some of the latter making him curious as to their value.

Back with the City of New York in 1937 he was Division Engineer in charge of the Laboratory and Inspection Division of the Board of Water Supply, an organization engaged in the building of the 400 million dollar Delaware Aqueduct, including three large earth dams, 110 miles of tunnel and pertinent operating structures.

He is a member of the A.S.C.E. and the Municipal Engineers, has written a discussion in the *Proceedings* of the former, some articles in *Rock Products*, the A.S.T.M. *Bulletin* and several in his favorite magazine the *Delaware Water Supply News*, published by the Board of Water Supply. He taught structural design at night at Pratt Institute in Brooklyn for five years and considers his most worthwhile work outside of engineering his 10 years as Scoutmaster and Scout Commissioner. Much pleasure is derived at present from volunteer work for the Traveler's Aid Society where he can spout Dutch to Hollanders just arrived, who are as bewildered as he was in 1906. Only one ambition remains—to skate backwards gracefully; his shape is against him.

F. E. Wolosewick

"Flexure of Cellular Shells," p. 249, is by F. E. Wolosewick, registered structural engineer, Chicago, Ill. Mr. Wolosewick graduated from Armour Institute of Technology and received his M. S. from Massachusetts Institute of Technology.

A Member of ACI, Mr. Wolosewick is a Charles Ellett Award Winner of the Western Society of Engineers. While associated with Universal Oil Products Co., oil refinery engineers, Chicago, he analyzed and designed piping systems subject to high temperatures and worked on special structural designs.

He is at present associated with Sargent and Lundy, engineers and designers of power plants.

L. A. Dahl

L. A. Dahl, Senior Research Mathematician for the Research and Development Department of the Portland Cement Assn., presents a condensation of Chapter 5, "Concrete Exposed to Sulfate Soils," of the Long-Time Study of Cement Performance in Concrete in "Cement Performance in Concrete Exposed to Sulfate Soils," p. 257.

Mr. Dahl is a graduate of Michigan State College and has been connected with the cement industry for nearly 30 years. From about 1920 to 1930 he was with Lehigh Portland Cement Co., at first directing diamond drill operations, and later in charge of the chemical division of the Central Laboratory. In 1929 he published a paper on the slide-rule for calculating compound composition of portland cement which is now in general use in the industry. Some of the equations presented in that paper appear in current specifications for portland cement.

From 1930 to 1934 Mr. Dahl was in charge of research at the California Portland Cement Co., and cooperated with the Hoover Dam Cement Investigation at Berkeley. He devised the methods used in designing raw mixtures for the experimental cements used in that investigation. Since that time he has been with the Portland Cement Assn., engaged principally in the statistical analysis of data and interpretation of phase-equilibria data. He was in charge of the Manufacturing Research Bureau of the association for a time, and has also worked on problems of phase equilibria at the association's fellowship at the National Bureau of Standards. He has written numerous papers pertaining to cement technology, and has published papers on proportioning cement raw materials and on the mathematical principles involved in phase equilibria studies.

Joseph J. Shideler and Wilbur H. Chamberlin

For the past two years, Joseph J. Shideler and Wilbur H. Chamberlin, Material Engineers, Research and Geology Division, U. S. Bureau of Reclamation, Denver, Colo., have been conducting tests to determine the effect of placing and curing temperatures on the compressive strength of concrete. Durability tests have also been coordinated with these studies. They present a portion of these studies in "Early Strength of Concrete as Affected by Steam Curing Temperatures," p. 273.

Mr. Shideler joined the Bureau of Reclamation in 1939 and has been associated with investigations of thermal and physical properties of concrete for Bureau structures, as well as with the electrical gaging, testing and control equipment used in the laboratory.

He received a B. S. in electrical engineering from the University of Denver in 1937. He is a member of ACI, A. I. E. E., Colorado Society of Engineers and is a registered engineer in Colorado.

Mr. Chamberlin also joined the Bureau in 1939 after serving with the Corps of Engineers, U. S. Army, for seven years. He was associated with the construction of the Fort Peck Dam and the Conchas Dam.

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While with the Bureau, he has been mainly interested in the testing of concrete used in massive structures to determine length change, flexural properties and the dynamic modulus of concrete under various curing conditions. Mr. Chamberlin received his technical training at Kansas State College and the University of Colorado.

V. Bogvad-Christensen

V. Bogvad-Christensen, structural engineer, City Engineer's Office, San Francisco, Calif., offers "A Shortcut for Determining Reinforcement in Reinforced Concrete" on p. 285.

Except for about three years work with the San Francisco-Oakland Bay Bridge, Mr. Bogvad-Christensen has been designing reinforced concrete structures for the City Engineer's Office since 1926.

He graduated in civil engineering in Copenhagen, Denmark, in 1911, receiving a scholarship to study in the United States. Associated with Christiani and Nielsen, Civil Engineers and Contractors, Denmark, in 1915, he then went to Singapore from 1916-21 with Swanson and Schested, Engineers and Contractors. He returned to Denmark and worked in railroad construction 1921-24. He came to the United States in 1924 and was the structural engineer for the Y.W.C.A. building in Honolulu, Hawaii.

Reviewers volunteer

In response to requests for reviewers of Italian technical publications, two JOURNAL readers have volunteered—Gennaro Mianulli, Brooklyn, N. Y., and Dr. J. J. Polivka, Berkeley, Calif. Mr. Mianulli is associated with Mianulli, Lamarca and Associates, Structural Engineers, Brooklyn. Dr. Polivka is a consulting engineer and has had articles published in several engineering magazines.

Clare H. Bullen

Clare H. Bullen, treasurer of the American Concrete Pipe Assn., Chicago, Ill., died recently. Mr. Bullen went to Yankton College, Yankton, S. D., and later to South Dakota State College, Brookings, S. D., graduating in 1898. He and his father founded the Portland Cement Products Co., Portland, Ore., in 1909, which expanded to 18 branches on the Pacific Coast. In 1926 Mr. Bullen sold his holdings on the West Coast, moved to Chicago and founded the Mid-West Concrete Pipe Co. An ACI member since 1938, he was president of the American Concrete Pipe Assn. in 1931-1932; director from 1924 to 1926 and 1933 to 1935; vice-president in 1942 and treasurer from 1944 to 1949.

John Tucker, Jr.

John Tucker, Jr., ACI member since 1926, died Nov. 20, 1949. Chief of the Concreting Materials Section, Mineral Products Division of the National Bureau of Standards since 1926, he actively directed the research and testing of cement, aggregates and other concreting materials.

Mr. Tucker had published several papers dealing with cement and concrete including "The Physical Properties of Cast Stone," ACI JOURNAL, Dec. 1931. He is well known for his work on statistical theory of strength, particularly as applied to concrete beams. He received the A. S. T. M. Sanford E. Thompson Award in 1942 for his paper "Statistical Theory of the Effect of Dimensions and of Method of Loading upon the Modulus of Rupture of Beams." At the time of his death, four papers dealing with the statistical theory of strength and with the properties of concrete were awaiting publication.

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Positions and Projects — ACI Members

Transportation and traffic committee includes ACI Members

The Institute of Transportation and Traffic Engineering, University of California, Berkeley, Calif., has appointed an enlarged Advisory Committee which includes four ACI Members.

The four men are: Wallace B. Boggs, County Surveyor and Road Commissioner, Alameda County, Oakland, Calif.; Roy W. Crum, Director, Highway Research Board, Washington, D. C.; Harmer E. Davis, Director, Institute of Transportation and Traffic Engineering, Berkeley; and R. M. Gillis, California Assistant State Highway Engineer, Sacramento, Calif.

The new committee, consisting of 17 original members and two new members, was appointed for terms ranging from one to three years so as to arrange for a staggered turnover in membership.

75th anniversary of the Norwegian Society of Engineers

The Norwegian Society of Engineers celebrates its 75th anniversary this year. Founded as the Norwegian Society of Engineers and Architects on December 9, 1874, its basic aim was to create a forum for specific professional discussions and to work for a better lay understanding of the importance of the technical and architectural professions. In 1912 the society divided and the architects formed their own national association and the engineering group adopted the title "The Norwegian Society of Engineers."

The society is composed of 37 local branches scattered throughout Norway. The large professional bodies such as the railway and road engineers, the engineers in the mining and saltpeter industries and the logging engineers have their own special branches. By far the largest branch is Oslo, which at present includes 12 professional sections.

The society's membership has grown steadily since it was founded by the nine charter members and now numbers nearly 5000, 2000 of whom belong to the Oslo branch. Eligibility for membership is restricted, in general, to engineers who have graduated from the Norwegian Institute of Technology or from foreign institutes of equivalent standing. There is no gradation in membership.

During the German occupation of Norway from 1940 to 1945 the Nazis dismissed the society's legal administrators and replaced them by a Nazi commissar. All loyal members at once withdrew their membership and an underground administration was formed. Norwegian engineers played an honorable and valuable part in the resistance movement. Many were sent to concentration camps, and others escaped to foreign countries. Not a few of them gave their lives in the fight for Norway's freedom.

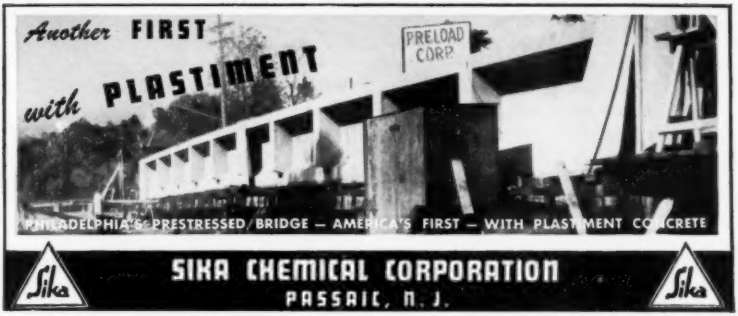
Since 1945 the society as a whole and its members individually have been mainly occupied with the problems of national reconstruction.

Loney retires

N. M. Loney, industrial consultant, Detroit, Mich., retired recently from active practice. The author of several papers in the ACI JOURNAL, Mr. Loney had also been active in Institute affairs. He was a member of the Advisory Committee, 1922-26; Board of Direction, 1924-25; chairman of Committee C-6, Field Methods; and chairman of Committee C-2, Concrete Floor Finish.

He graduated in mechanical engineering from Armour Institute and held various positions until 1906 when he was appointed chief engineer for the American Can Co. In 1919 he became vice-president in charge of industrial construction for the Thompson Starret firm.

Mr. Loney joined the Fisher Body Division of General Motors Co. in 1925,



holding various engineering and management positions, including Director of Works Engineering, until 1947. Since then he maintained an office as industrial consultant.

While with the American Can Co., Mr. Loney made some of the first strain tests on fully loaded flat slabs.

International housing competition extended

The Instituto Técnico de la Construcción (Technical Institute of Construction), Madrid, Spain, has extended the closing date for their international competition from November 15, 1949, to March 15, 1950.

An award of 100,000 pesetas (about $9,000) will be given for the best project of industrial development necessary to manufacture machinery, accessories and materials to build dwellings to house 50,000 Spanish families each year. The competition was conceived by this non-profit organization to alleviate the critical housing shortage in Spain.

Fowle appointed assistant city engineer

Royal E. Fowle, formerly vice-president and manager of the Granite Rock Co., Watsonville, Calif., has been appointed assistant to H. B. Kitchen, city engineer of Watsonville, who is planning to retire the first of the year. Mr. Fowle will then assume the duties of city engineer, superintendent of streets and sewers, and water department manager. Mr. Fowle served for many years as civil engineer and manager of the Logan, Calif., plant of Granite Rock Co. before he was appointed vice-president and manager. He attended Ohio State University and the California Institute of Technology. He is a member of ACI, A. S. C. E., American Institute of Mining and Metallurgical Engineers and American Society for Testing Materials.

Couse nominated for A. G. C. presidency

Associated General Contractors of America has nominated Walter L. Couse, president of the Walter L. Couse Co., Detroit, Mich., as president, succeeding Adolph Teichert, Sacramento, Calif. Nomination is tantamount to election and officers will be installed at the 31st annual convention in San Francisco, Feb. 27-Mar. 2, 1950. Glenn W. Maxon, Dayton, Ohio, was nominated to succeed Couse as vice-president.

Ernst and Dunmire form partnership

George C. Ernst, ACI Member since 1935, and E. H. Dunmire have formed the partnership of Dunmire and Ernst, Consulting Engineers, Lincoln, Neb. The partnership will specialize in structures, waterworks, sewage treatment, irrigation, electrical systems, and appraisals. Mr. Ernst retains his association with the

Continued on p. 22

We Want to Know—

ACI Members will recall that the membership classification forms circulated last year contained a question—"What suggestions have you for the Technical Activities Committee as to papers or reports you'd like to see published; subjects you'd like to see discussed?" This is the fourth group of such questions published, the first group appearing in the September *News Letter*.

One thing that will undoubtedly develop from the many replies is a special session at the Institute's convention next February at which a panel of "experts" would head the discussion.

Meanwhile the ACI JOURNAL will publish anonymously a good many of these questions and suggestions. JOURNAL publication is intended to bring responses which may be the answer or partial answer to questions asked. Readers are not only invited to answer those questions appearing in print, but to send other suggestions and queries as well. Some of these questions have been answered in past ACI literature and will be covered more thoroughly in future issues of the JOURNAL Letters from Readers sections.

—Editor

Compression test

Is the compression test for determining the quality of concrete obsolete?

Retaining walls

Papers on concrete retaining walls would be of interest. We recently consulted eight different professional engineers on a specific retaining wall problem —eight entirely different answers.

Paint reaction

Is there chemical reaction between concrete in pavements and traffic marking paint?

Hooked bars

Are hooked bars worth the cost of bending?

Standardization of specifications

I am very much interested in standardization of specifications for cement, aggregates, concrete products and concrete design. I think that ACI should continue and enlarge upon its efforts in this field and continue to take an active and aggressive program in these fields together with such organizations as the American Standards Assn., A.S.C.E. and A.S.T.M.

Limit design

I would like to see more papers on "limit design" based upon the plastic theory rather than the elastic theory.

The general contractor

How about more articles of interest to the general contractor?

Radiant heating

More papers and reports covering in detail the uses of concrete in radiant heating are desired.

Strength and fineness modulus

What is the relation of compressive strength to fineness modulus in coral aggregates?

Practical engineering

I would suggest that more papers be devoted to the practical engineering applications and a smaller percentage to basic theoretical considerations.

Bond

What is the effect of air entrainment on bond?

Construction

I would enjoy seeing articles on how some particular concrete problem—not design—but field or construction was solved.

Size gradation

What is the effect of particle size gradation on the properties of portland cement and concrete?

Continued on p. 22

Honor Roll

February 1, 1949, to October 31, 1949

The current Honor Roll comes to a final tally January 31, 1950.

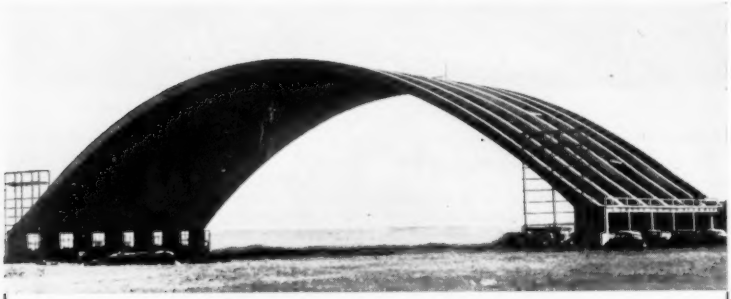
The number of Members, to date, who have proposed one or more new Members since last February represents 4 percent of our total membership. If each of the Members who comprise the remaining 96 percent should introduce but one new Member to ACI we'd soon double our membership.

"Morgan and Company" continues to roll along and has a total of 81½ credits on our Honor Roll in the two top spots.

It seems fitting, however, to note that our membership grows by less spectacular achievement in the realm of our Members' "Member Consciousness." The next 26 names on the Honor Roll through their several efforts are responsible for 106 more new Members. Then add the groups in which each Member has been responsible for one or two Members or a joint sponsor in member effort. If a hurried tally is correct then the remaining "Honor Rollers" on the list introduced 274½ new Members.

Thus the tally: 341 member conscious Members have added more than 450 new Members. Each of the 341 has enabled ACI to do more for all its 5000 and we hope and believe has done a favor to each of the 450.

Newlin D. Morgan (Ill.)	54½
Newlin D. Morgan, Jr. (Wyo.)	27
F. N. Menefee (Mich.)	11½
Jose Antonio Vila (Cuba)	10
R. H. Sherlock (Mich.)	9
Jaime Alberto Mitrani (Cuba)	6
H. J. Gilkey (Ia.)	5½
James A. McCarthy (Ind.)	5
J. Vicente Orozco (Mexico)	4
John J. Hogan (N. Y.)	3½
William T. Neelands (Ga.)	3½
Frank E. Richart (Ill.)	3½
Thomas E. Stanton (Calif.)	3½
Raymond E. Davis (Calif.)	3
H. D. Jolley (Ill.)	3
Barton Jones (Puerto Rico)	3
J. W. Kelly (Calif.)	3
Frederic F. Mavis (Pa.)	3
Leon Venegas (La.)	3
Stanton Walker (D. C.)	3
Roberto Barillas F. (Guatemala)	2½
Harold S. Carter (Utah)	2½
R. A. Caughey (Ia.)	2½
Roger H. Corbetta (N. Y.)	2½
H. F. Gonnerman (Ill.)	2½
Ralph W. Kluge (Ind.)	2½
A. J. McElrath (Tenn.)	2½
Howard Simpson (Mass.)	2½
Paul W. Abeles (England)	2
A. Amirikian (Md.)	2
J. F. Baxter (N. Y.)	2
Sterling Lowe Bugg (Fla.)	2
W. S. Cottingham (Wis.)	2
John G. Dempsey (N. Y.)	2
Ambrosio R. Flores (Calif.)	2
Zorislav Franjetic (Argentina)	2
Henry E. Griset (N. C.)	2
Robert J. Hansen (Mass.)	2
V. P. Jensen (Calif.)	2
Emory E. Johnson (S. D.)	2
R. R. Kaufman (Ohio)	2
Henry L. Kennedy (Mass.)	2
Karl W. Lemcke (N. Y.)	2
Sam McCluer (Ga.)	2
William S. Moore (Ind.)	2
George G. Nichols (Ore.)	2
William D. Nowlin (Va.)	2
Luis Paredes-Manrique (Colombia)	2
Luis A. Pietri-Lavie (Venezuela)	2
Dan H. Pletta (Va.)	2
Thomas C. Shedd (Ill.)	2
G. E. Troxell (Calif.)	2
George Winter (N. Y.)	2
S. J. Chamberlin (Ia.)	1½
Jacob J. Creskoff (Pa.)	1½
H. C. Delzell (Ill.)	1½
A. H. Douglas (Canada)	1½
Adly W. Gindy (Egypt)	1½
Adli Milik Hanna (Egypt)	1½
Elmo C. Higginson (Colo.)	1½
Edgar R. Kendall (Nebr.)	1½
Robert L. Mauchel (N. Y.)	1½
H. J. McGillivray (Fla.)	1½
Harmon S. Meissner (Colo.)	1½
Carl A. Menzel (Ill.)	1½
Edward W. Scripture, Jr. (Ohio)	1½
Roy T. Sessums (La.)	1½
Bertrand H. Wait (N. Y.)	1½
Calvin T. Watts (La.)	1½
Frank W. Wheeler (Va.)	1½
Jerome O. Ackerman	1
William F. Anderson	1
Samuel Baker	1
J. A. Bakker	1
Charles F. Ball	1
John H. Bateman	1
Donald C. Beam	1
Oscar Benedetti	1
H. J. Bezette	1



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Carlos Mora Bravo	1	Gerald K. Gillan	1
Milton Brumer	1	Castor Segunda Goa	1
R. Y. Bush	1	Ali N. Gokyigit	1
Adraino Carbone	1	E. Gonzalez-Rubio	1
D. R. Cervin	1	Armour T. Granger	1
R. V. Chate	1	Samuel J. Green	1
H. Clare, Jr.	1	K. Hajnal-Kon	1
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Ray A. Crysler	1	J. P. Jain	1
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Peter J. Doanides	1	Milton Kellert	1
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Harlan H. Edwards	1	George E. Large	1
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Ewell W. Finley	1	Leslie D. Long	1

Winthrop E. Luke 1
E. Lyons 1
Charles Macklin 1
Ian MacTaggart 1
Leon C. Magbanua 1
Glenway Maxon 1
George C. McLeod 1
F. R. McMillan 1
M. J. McMillan 1
Jesus Mesa 1
Robert B. B. Moorman 1
Robert Morris 1
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Orin G. Patch 1
Jose F. Perez 1
Anthony F. Pessolano 1
Charles Peterson 1
A. Edward Piller 1
James M. Polatty 1
W. S. Powell 1
Emil Praeger 1
Gayle B. Price 1
Raul Miguel Rivero 1
Georges Ronai 1
Albert Rootberg 1
John A. Ruhling 1
Carl E. Sandstedt 1
Emil Schmid 1
Philo H. Schultz 1
F. Selmer 1
Saul Shaw 1
Isaac S. Shina 1
Ranbir Singh Shrikant 1
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Luis G. Solano A. 1
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Leroy A. Staples 1
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Martin Steljes 1
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E. O. Sweetser 1
Manabu Tanaka 1
Maurice Taylor 1
T. W. Thomas 1
I. L. Tyler 1
Oscar J. Vago 1
Joseph J. Waddell 1
John M. Wells 1
J. G. Welsh 1
Donald K. Woodin 1

The following credits are, in each instance "50-50" with another member.

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Birger Arneberg
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C. Harold Barcus
Charles H. Barker
Hugh Barnes
Frank Baron
J. F. Barton
J. E. Bates
Aaron H. Bauman
E. W. Bauman
Fred Beatty
H. W. Beckingham
A. H. Brewer
Harold W. Brewer
A. R. Brickler
A. A. Brielmaier
Robert G. Burnett
Henry B. Campbell
J. A. Carrick
Napoleon V. Campomanes
W. R. Chandler
Arthur P. Clark
Leo Coff
T. F. Collier
R. D. Conrow
W. A. Coolidge
Dwight A. Covington
William A. Crabb
Lauro M. Cruz
A. E. Cummings
M. H. Cutler
W. A. Davis
Curzon Dobell
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Jay C. Ehle
W. J. Emmons
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Benjamin Forsyth
James J. Fox
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M. J. Hawkins
John M. Hayes
James A. Hedges
James A. Highsmith, Jr.
A. E. Hiscox
Myle J. Holley, Jr.
S. C. Hollister
Albert C. Hooke
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Melvin Fu Hoon Ja
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M. E. James
Bruce Jameyson
Bruce M. Johnson
Samuel Judd
Lane Knight
Carl A. Koerner
James V. Mandia
Henri Marcus
Hudson Matlock
J. W. McBurney
Robert B. McCalley, Jr.
T. R. McCullough
Jack Robert Miller
Eugene Mirabelli
Eric C. Molke
Sylvester Morabito
Wm. Morris
William H. Munse
S. Neatwait
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George L. Otterson
Chester B. Palmer
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Joe Peeler Patrick
Robert J. Perrich
George Nelson Perry
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Warren Raeder
Irvin S. Rasmusson
F. V. Reagel
Allan M. Reese
Raymond C. Reese
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C. D. Riddle
Paul Rogers
William H. Rowan
Fred H. Ryan
Cecil F. Schaaf
Leo W. Schmidt
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Tirlochan Singh
Myron M. Smith
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Homer Thrall
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Fehiman Tokluoglu
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John Tucker, Jr.
Maxwell Upson
F. E. Votaw
Norman Wagner
W. Wayne Wallace
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Frederic N. Weaver
Carl Weber
J. Wegrowski
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H. F. Williams
Paul C. Winters
M. O. Withey
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K W Wright
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Alexander H. Yeates
R. B. Young
Robert Zaborowski

Are
YOU
"Member
conscious"?

New Members

Of the 68 applications approved for membership for the month of October there were 32 Individual, 7 Junior and 29 Student.

Illinois was the leading membership state for the month providing 10 new Members. Massachusetts was close behind with 7 new applicants.

Our membership total October 31, 1949, became 4834.

California

Carter, Robert W., (Indiv.) Civil Engineer, 2820 Monroe St., San Diego 4, Calif.
Glenn, G. D., (Indiv.) 3703 F St., Sacramento, Calif.
Hawke, James P., (Indiv.) 111 Windsor Ave., Berkeley 8, Calif.
Porter, John E., (Indiv.) Metz Aggregate Co., P. O. Box 900, Watsonville, Calif.

Colorado

Falkenberg, John F., (Jr.) 2000 E. 7th Ave., Denver 6, Colo.

Georgia

Matzenbach, Waldo H., (Indiv.) Rt. No. 1, Martinez, Ga.
Taylor, Thomas Frederic, (Indiv.) P. O. Box 658, Augusta, Ga.

Illinois

Arner, Richard Dale, (St.) 805 S. Lincoln, Urbana, Ill.
Burger, William J., (St.) 10221 Prairie Ave., Chicago 28, Ill.
Farr, Howard J., (St.) 2103 S. 20th Ave., Maywood, Ill.
Hickman, Charles Lambert, (St.) 1106 W. Illinois, Urbana, Ill.
Kessler, Howard Edwin, (St.) 1215 S. Fourth St., Champaign, Ill.
Reihmer, George Wesley, (St.) 1024 Hayes Ave., Oak Park, Ill.
Roman, Charles L. Jr., (Jr.) 1710 Crain St., Evanston, Ill.
Ruscitti, Ray Salvatore, (St.) 1138 S. Troy St., Chicago 12, Ill.
Ticer, James E. Jr., (St.) c/o Box 87, Salem, Ill.
Wolosewick, F. E., (Indiv.) 4029 W. Warwick Ave., Chicago 41, Ill.

Indiana

Binler, Ilyas Elfiye, (Jr.) 333 Stadium Ave., W. Lafayette, Ind.
Fan, W. Y., (St.) c/o Mr. I. S. Tsai, 219 S. Grant, W. Lafayette, Ind.
Laskowski, John J., (St.) Box 419—Apt. 13C, Notre Dame, Ind.
Woods, James Darse, (St.) 1004 Churchman Ave., Indianapolis, Ind.

Iowa

Davis, Dean Allen, (St.) 1586 Wood St., Dubuque, Ia.
Patterson, Wesley H., (St.) 1102 Park Lane, Knoxville, Ia.

Louisiana

Belden, R. L., (Indiv.) 2421 Hanson St., Alexandria, La.

Maine

Sanborn, Leonard F., (Indiv.) 9 Green St., Augusta, Me.

Massachusetts

Biggs, John Melvin, (Indiv.) Room 1-251, MIT, Cambridge, Mass.
Brown, Burtis, (Indiv.) 127 Federal St., Boston 10, Mass.
Dunkerley, Paul Albert, (Indiv.) 372 Metropolitan Ave., Roslindale 31, Mass.
Guvenc, Bozkurt, (St.) The Graduate House, MIT, Cambridge 39, Mass.
Harvey, Francis S., (Indiv.) 141 Dewey St., Worcester 2, Mass.
Vale, Charles W. Jr., (Indiv.) Apt. 51, 222 Marlborough St., Boston, Mass.
Young, Stephen Y. C., (St.) Box 115B, MIT, Graduate House, Cambridge, Mass.

Michigan

Sherzad, Ihsan M., (St.) International Center, Ann Arbor, Mich.
Shih, Tsze-sheng (St.) 603 E. Ann St., Ann Arbor, Mich.

Missouri

Tao, Kwang-yuen, (Jr.) Box 120, Washington University, St. Louis, Mo.

Montana

Kuetzing, Walter, (Indiv.) Box 864, Billings, Mont.

Nebraska

Gray, William Marion, (Indiv.) 1114 2nd Corso, Nebraska City, Nebr.

New York

Chen, Tien You, (St.) 546 W. 124th St., Apt. No. 31, New York 27, N. Y.
Meyer, Marvin Morton, (St.) 38 Northview Terrace, Rochester, N. Y.

Pao, Channing, (Indiv.) Dept. of Civil Engineering, College of Applied Science, Syracuse University, E. Syracuse 4, N. Y.
Wang, David T. C., (St.) YMCA, 55 Hanson Place, Brooklyn 17, N. Y.

North Carolina

Huske, Benjamin R. III, (Jr.) 522 Hawthorne Lane, Charlotte, N. C.

Ohio

Barber, C. Merrill, (Indiv.) Barber & Magee, Engineers, 1303 Prospect Ave., Cleveland 15, Ohio

Oregon

Caufield, James D., (Jr.) 2386 W. Burnside, Portland 5, Ore.
Johnson, Clifford Booth, (Indiv.) P. O. Box 82, McNary, Ore.

South Dakota

Everson, Helmer E., (Indiv.) 115 S. Monroe, Pierre, S. D.

Texas

Hammons, Doil S., (Indiv.) Box 256, College Station, Texas

Washington

Dobson, Richard T., (Jr.) 106 N. Dorm, Coulee Dam, Wash.
Wrigley, James B., (Indiv.) P. O. Box 698, Richland, Wash.

Wyoming

Charlson, Ancher Andrew, (Indiv.) U.S. Soil Conservation Service, Sundance, Wyo.

Australia

Hodge, William James, (Indiv.) c/o Standard Portland Cement Co., Ltd., Charbon, N. S. W., Australia

Canada

Bleackley, Frank J., (Indiv.) Apt. 21, 1839 Lincoln Ave., Montreal, Que, Canada
Duff, G. L., (St.) 16 Binscarth Rd., Toronto, Ont., Canada
Fournier, Rene, (Indiv.) Quebec Testing Laboratory, 13, Place d'Aiguillon, Quebec, Que., Canada
Hickey, W. E., (Indiv.) c/o The Foundation Co. of Ontario Ltd., 1220 Bay St., Toronto, Ont., Canada

Colombia

Mesa A., Gustavo, (Indiv.) Tenorifa 52-9, Medellin, Colombia

Cuba

Abalo, Miguel Angel, (St.) 22 No. 312—Vedado, La Habana, Cuba
Arvesu, Manuel, (St.) E. Barnett 55, Havana, Cuba
Leon, Federico (St.) Calzada 1157, Vedado, Havana, Cuba
Martin, Iganacio (St).) 462 San Miguel St., Havana, Cuba
Plotnik, Alberto, (St.) Jesus Maria No. 353, Havana, Cuba

Guatemala

Cruz Molina, Carlos Rafael, (St.) 3a Av. N. No. 57, Guatemala City, Guatemala, C. A.

Hawaii

Baker, M. F., (Indiv.) 3337 Maunaloa Ave., Honolulu, T. H.

Panama

Noriega, Tomas Noriega, (St.) Apartado No. 60, Panama, Panama

Philippines

de la Serna, Felix C., (Indiv.) Asst. Civil Engineer, Office of the District Engineer, Davao City, Philippines
Lee, Seng Lip, (St.) 225 Valenzuela, Santa Mesa, Manila, Philippines

Siam

Dhanuckamin, Somthob, (Indiv.) 912 B. Klong Banggikhan, Dhonburi, Siam

Syria

Omari, Shakib, (Indiv.) Noori Pasha St., Salhia, Damascus, Syria

Positions and Projects

Continued from p. 15

University of Nebraska as Professor and Chairman of the Department of Civil Engineering.

Dennis retires

T. H. Dennis ended a 37-year career with the California State Division of Highways when he retired as maintenance engineer on Sept. 30. Under his leadership the state has spent more than $3,000,000,000 on highway maintenance in the last 23 years in addition to about $3,000,000 a year from construction funds handled through his department. Mr. Dennis promoted the use of radio in highway departments, and as chairman of the American Association of State Highway Officials' committee on radio use, he was instrumental in getting the FCC to reserve a group of frequencies solely for the use of highway departments.

J. L. Savage in New Zealand

J. L. Savage, consulting engineer of Denver, Colo., and widely known for his work in hydraulic structures, is currently occupied on engineering assignments in New Zealand where he is investigating hydro-electric developments. He will particularly study the proposals for the Roxburgh Gorge project in the South Island. This development will have an ultimate installed capacity of 320,000 kw. This is being followed by an assignment for the government of New South Wales in Australia.

We Want to Know

Continued on p. 16

Inspection

The JOURNAL should stress the importance of having rigid inspection in the manipulating of concrete. A top-notch engineer with a score of years of concrete experience belongs on all jobs.

Static and dynamic strains

What are the methods of measuring static and dynamic strains in concrete structures, especially with respect to earthquake effects?

Shrinkage in rich mixes

One of our major problems is how to overcome shrinkage in the setting of rich mixes of concrete. A study of how to overcome this problem would be of great interest to us.

Stirrups

What is a simple and *rapid* method for determining the stirrup requirements for reinforced concrete beams?

Prepack

More information is desired on the use of prepacked concrete in *new* construction.

Long life

Concrete structures do not have the life that they should have. How can this be improved?

Simplified formulas

The data given in the ACI Code for moment distribution of two-way rectangular slabs require much study and frequent usage for its mastery. These code provisions permit many misinterpretations by the average engineer. Simplified formulas would be desirable.

Simplified formulas are needed for positive and negative stresses in eccentrically loaded columns (concrete, concrete with steel cores and steel cores alone) each predicated upon fixed, semi-fixed and hinged ends. Included with these there should be formulas for locating the points of contra-flexure in such members.

Current data for computing moments in rigid frames are cumbersome and offer little chance for visualization and a step-by-step check of one's computations

High lifts

What is the effect of high lifts (20 to 40 ft) in dam construction?

Entrained air

What are the effects of vibration, pumping (pumpcrete) and other handling on entrained air in concrete?

Registration List—New England Regional Meeting

Asterisks (*) denote ACI Members

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ACI publications in large current demand

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

Approved as a Standard of the American Concrete Institute at its 44th Annual Convention February 25, 1948. Ratified by letter ballot June 29, 1948. It is of large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$3.00, to ACI Members—$1.75.

ACI Standards—1949

218 pages, 6x9 reprinting current ACI Standards: Building Code Requirements for Reinforced Concrete (ACI 318-47); Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46), six recommended practices: Use of Metal Supports for Reinforcement (ACI 319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); Construction of Concrete Farm Silos (ACI 714-46); Winter Concreting Methods (ACI 604-48); Application of Cement Paint (ACI 616-49); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—ten ACI Standards in one book, $2.50 per copy—to ACI Members, $1.50. The "Detailing Manual" (ACI 315-48) is available separately only at its established price—see above.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944, $1.25 per copy, 75 cents to members.

Air Entrainment in Concrete—Book 2 (1947)

A 204-page compilation of information on air entrainment published after Book 1 (1944), including: "Field Use of Cement Containing Vinsol Resin", and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures", by Charles E. Wuerpel, "Entrained Air in Concrete", a foreword and 14 short papers presented at the 1946 Convention; and discussion of the symposium, reprinted from ACI JOURNALS for September 1945, Feb., June and Dec. Part 2, 1946. $2.25 per copy, $1.50 to ACI Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940, 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

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SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 21 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES (ACI 616-49) 46-1

Price 50 cents (in special covers).

REPORT of COMMITTEE 616—Sept. 1949, pp. 1-16 (V. 46)

Supersedes 38-30 and 45-18.

This ACI standard establishes recommended practices for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage, and general characteristics and factors affecting durability.

AN ULTRASONIC METHOD OF STUDYING DETERIORATION AND CRACKING IN CONCRETE STRUCTURES.....46-2

Price 35 cents.

J. R. LESLIE and W. J. CHEESMAN—Sept. 1949, pp. 17-36 (V. 46)

A new method and apparatus for field and laboratory testing of concrete is described. The apparatus called the "Soniscope" was originally designed to detect internal cracks in concrete. It develops pulses of ultrasonic sound in the material and measures the velocity of their transmission through it. This pulse velocity has the unique advantage of being independent of the size or shape of the body under test. Measurements can be made with equal facility in mass concrete, slabs or laboratory specimens.

The existence and extent of internal cracks and the depth of visible surface cracks can be determined by the use of this apparatus.

The velocity has been found, by experiment, to be a reliable measure of the condition of the concrete and is particularly useful in deterioration studies. The dynamic modulus can be calculated from this velocity, and values so obtained are found to agree closely with the results of tests using established methods.

MANUFACTURE OF REINFORCED FOAM CONCRETE ROOF SLABS ...46-3

Price 35 cents.

I. T. KOUDRIASHOFF—Sept. 1949, pp. 37-48 (V. 46)

The Russian type of lightweight concrete described used a rosin-glue emulsion to preserve the air voids before the initial set of the cement. Shrinkage was decreased and strength increased through high pressure steam curing. The autoclave treated foam concrete used in the production of precast industrial roof slabs had a unit weight of 47 lb per cu ft and a compressive strength of over 500 psi. The lightweight slabs, used in a load carrying capacity and as insulation, reduced construction time by 50 percent and costs by as much as 20 percent. Test data on roof slabs and production procedures are also described.

SUGGESTIONS ON CONCRETE FLOOR CONSTRUCTION 46-4

Price 35 cents.

ERNST GRUENWALD—Sept. 1949, pp. 49-56 (V. 46)

The relationship between good concrete floors and the proper selection of cement and aggregates is discussed. Data are cited to emphasize the advantage of coarse-aggregate mixes over cement-sand topping for concrete floors.

USE OF AIR-ENTRAINING CONCRETE IN CANAL LINING 46-5

Price 35 cents.

JOSEPH J. WADDELL—Sept. 1949, pp. 57-64 (V. 46)

This paper presents a discussion based on field observations of the use of air entrainment in canal lining concrete. The Bureau of Reclamation made studies of air-entraining agents for use in the irrigation canal lining for the Friant-Kern Canal on the Bureau's Central Valley Project in California. Results to date indicate that appreciable benefits accrue when an air-entraining agent is used in concrete which is placed and compacted by means of a mechanical slip-form. Care is necessary in adjusting concrete mixes to incorporate entrained air because of the sensitivity to mix changes of concrete for slip-form placement.

THE USE OF PORTLAND-POZZOLAN CEMENT BY THE BUREAU OF RECLAMATION..46-6

Price 35 cents.

ROBERT F. BLANKS—Oct. 1949, pp. 89-108 (V. 46)

The Bureau of Reclamation has made extensive studies of pozzolanic materials, and portland-pozzolan cements are now being used in many of the major structures built by the Bureau. The properties of portland-pozzolan cement that are used advantageously in the production of mass concrete are described.

RESISTANCE OF CONCRETE AND PROTECTIVE COATINGS TO FORCES OF CAVITATION.....46-7

Price 35 cents.

WALTER H. PRICE and GEORGE B. WALLACE—Oct. 1949, pp. 109-120 (V. 46)

A machine used for producing cavitation erosion in the laboratory is described and the results of tests made to investigate the effect of mix proportions, surface treatment, and protective coatings on the resistance of concrete to cavitation are discussed. Through proper use of these, the resistance of concrete surfaces to cavitation erosion may be extended three or four times, but even the best concrete will not resist the forces of cavitation for a prolonged period. Heavy rubber coatings bonded well to the surface of the concrete have proved effective.

VACUUM PROCESSES APPLIED TO PRECAST CONCRETE HOUSES...46-8

Price 35 cents.

K. P. BILLNER and BERT M. THORUD—Oct. 1949, pp. 121-128 (V. 46)

The use of vacuum processes in precast concrete construction simplifies the building of fire-resistant, durable structures designed to permit the maximum use of like units. Vacuum processes have thus far been used in one- and two-story houses and one-story industrial or warehouse structures, but further developments should make possible similar benefits for multi-story construction.

The specific vacuum methods utilized are: (1) extracting excess water from freshly placed concrete prior to set, thereby increasing early strength and enabling early handling of units, (2) holding forms in place by vacuum and (3) handling and placing finished and hardened concrete units by cast-in-place closures formed and quickly hardened by vacuum processes.

The use of these methods results in high strength monolithic construction having exterior and interior finished surfaces, with insulation incorporated into the construction, if desired.

NEW TYPE OF CONSISTENCY METER TESTED AT ALLATOONA DAM....46-9

Price 35 cents.

JAMES M. POLATTY—Oct. 1949, pp. 129-136 (V. 46)

Various mechanical devices have been developed for indicating consistency of concrete since the early days of visual inspeciton after discharge from the mixer. A new type meter to measure the consistency of concrete while it is being mixed was tested at Allatoona Dam and the operation and results are described.

EFFECT OF MIXING SEQUENCE ON THE PROPERTIES OF CONCRETE....46-10

Price 35 cents.

F. L. FITZPATRICK and W. SERKIN—Oct. 1949, pp. 137-140 (V. 46)

The order in which the constituents of concrete (aggregates, cement and water) are combined in the mixing operation, has a significant effect upon the properties of the concrete as to workability, strength, density, surface finish and absorption. Tests are reported.

BOND OF CONCRETE REINFORCING BARS....46-11

Price 35 cents.

ARTHUR P. CLARK—Nov. 1949, pp. 161-184 (V. 46)

The tests reported were made to compare the resistance to slip in concrete (bond) of deformed bars when tested in beams and companion pull-out specimens, to secure information on the effects of size of bar, the type of deformations on the bars and the strength of concrete on the bond. The bars were cast in a horizontal position in all test specimens. The variables were depth of concrete under the bar, length of embedment of the bar in the concrete, strength of concrete and diameter of bar. Slip of the bar was measured at the loaded and free ends. Three tests were made with 2 in. of concrete under the bar, with 15 in. of concrete under the bar and with 3-in., 12-in., and 16-in. embedments.

Bond strengths for the beams and the pull-out specimens were affected similarly by changes in the geometry of the bars and the bond test specimens. They were greater when the bars were near the bottom than when they were near the top of the specimens. The highest bond strengths were obtained with bars having deformations conforming to suggested requirements for maximum spacing and minimum height and providing ratios of shearing to bearing areas less than 10, usually less than 6.

PERLITE AGGREGATE: ITS PROPERTIES AND USES....46-12

Price 35 cents.

J. JOHN BROUK—Nov. 1949, pp. 185-192 (V. 46)

Synthetic expanded volcanic rock, better known as perlite is a fairly recent addition to the lightweight aggregate field. Its use in concrete is governed by weight, gradation, mixing procedure, cellular structure, strength of cell walls, insulating properties, cost, etc. Air entrainment appears to be necessary to give a workable, nonsegregating mix. Perlite aggregate blended with sand can be used in high strength structural concrete and concrete products.

THIN WALL CONCRETE SHIP CONSTRUCTION....46-13

Price 35 cents.

FRANCIS R. MAC LEAY—Nov. 1949, pp. 193-204 (V. 46)

The development of construction methods for placing thin concrete walls is described. After several attempts using the standard method of pouring between double forms, a new method was developed which permitted the successful construction of ¾-in. and 1½-in. concrete walls. Tests were also conducted to determine the practicability as well as the strength of Gunite as a medium to unite precast units into a monolithic ship. Later these methods were used successfully in the construction of a concrete landing craft for the United States Navy.

SPECIFICATIONS SHOULD BE REALISTIC....46-14

Price 35 cents.

HARRY F. THOMSON—Nov. 1949, pp. 205-220 (V. 46)

Specifications for concrete in moderate-sized and lesser construction frequently contain provisions which are ambiguous, conflicting in application, or do not fully recognize local materials. Most of these questionable features result from (1) inadequate information regarding the characteristics of concrete, or (2) use of ready-written specifications without revision for conditions or changes in standard requirements.

Among the features discussed are: specifying both method and result; "frozen" specifications; habitual use of "1-2-4", multiple provisions for quality; strength without naming consistency; recognition of local materials; use of compression tests; time of placing; bin-test of cement. Numerous quotations from specifications are given, and suggestions offered for bringing the provisions in line with actual conditions.

INSPECTION AND TESTING OF MATERIALS....46-15

Price 35 cents.

NICOLAAS T. F. STADTFELD—Dec. 1949, pp. 237-248 (V. 46)

This paper sets forth the care taken in the inspection and testing of 7 million barrels of portland cement used in 4 million cu yd of concrete of great uniformity and every indication of durability after 12 years of observation. It stresses for the concrete the importance of low water solubility, freedom from "laitance," lack of cement burns of the workers, low alkali content, and it shows above all the necessity for manufacturing control of the cement clinker. It describes how premature set of concrete was prevented. It also deals with the inspection, grading, and testing of aggregates. The Board of Water Supply specifications for aggregates are given as well as other factors used in securing good material.

FLEXURE OF CELLULAR SHELLS...46-16

Price 35 cents.

F. E. WOLOSEWICK—Dec. 1949, pp. 249-256 (V. 46)

After a brief discussion of the uses of cellular shells and methods available for the solution of design problems, a sample problem is set up using the theorem of least work. From charts the moments are determined and the effect of added stiffeners is assessed.

CEMENT PERFORMANCE IN CONCRETE EXPOSED TO SULFATE SOILS....46-17

Price 35 cents.

L. A. DAHL—Dec. 1949, pp. 257-272 (V. 46)

The Long-Time Study of Cement Performance in Concrete deals with the performance of portland cements in concrete under various conditions of exposure in the field. Among these conditions is exposure to sulfate soils, that is, to the so-called "alkali soils." This part of the investigation is reported in Chapter 5, in which the complete data are given. The present paper describes briefly the work reported in Chapter 5 and the conclusions which have been drawn. Those readers who wish to study the results in greater detail are referred to the more complete report.

EARLY STRENGTH OF CONCRETE AS AFFECTED BY STEAM CURING TEMPERATURES.....46-18

Price 35 cents.

JOSEPH J. SHIDELER and WILBUR H. CHAMBERLIN —Dec. 1949, pp. 273-284 (V. 46)

The testing and results obtained on 990 6 x 12-in. concrete cylinders steam cured for various periods and at temperatures ranging from 100 to 200 F are discussed. Strength results are given for ages ranging from 6 hours to 28 days and strengths of companion specimens moist cured at 70 F are given whenever possible. The information presented was obtained for the Bureau of Reclamation for writing specifications covering the steam curing of precast units such as irrigation pipe.

A SHORTCUT FOR DETERMINING REINFORCEMENT IN REINFORCED CONCRETE.....46-19

Price 35 cents.

V. BOGVAD-CHRISTENSEN—Dec. 1949, pp. 285-292 (V. 46)

This paper presents a graph which gives the complete relationship between moments, thrusts, concrete dimensions, reinforcement and resulting stresses for the reinforced concrete members in question.

DISCUSSION

Discussion closes January 1, 1950

Sept. Jl. '49

Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)—Report of Committee 616

An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures—J. R. Leslie and W. J. Cheesman

Manufacture of Reinforced Foam Concrete Roof Slabs—I. T. Koudriashoff

Suggestions on Concrete Floor Construction—Ernst Gruenwald

Use of Air-Entraining Concrete in Canal Lining—Joseph J. Waddell

Discussion closes February 1, 1950

Oct. Jl. '49

The Use of Portland-Pozzolan Cement by the Bureau of Reclamation—Robert F. Blanks

Resistance of Concrete and Protective Coatings to Forces of Cavitation—Walter H. Price and George B. Wallace

Vacuum Processes Applied to Precast Concrete Houses—K. P. Billner and Bert M. Thorud

New Type of Consistency Meter Tested at Allatoona Dam—James M. Polatty

Effect of Mixing Sequence on the Properties of Concrete—F. L. Fitzpatrick and W. Serkin

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Nov. Jl. '49

Bond of Concrete Reinforcing Bars—Arthur P. Clark

Perlite Aggregate: Its Properties and Uses—J. John Brouk

Thin Wall Concrete Ship Construction—Francis R. Mac Leay

Specifications Should Be Realistic—Harry F. Thomson

Discussion closes April 1, 1950

Dec. Jl. '49

Inspection and Testing Materials—Nicolaas T. F. Stadtfeld

Flexure of Cellular Shells—F. E. Wolosewick

Cement Performance in Concrete Exposed to Sulfate Soils—L. A. Dahl

Early Strength of Concrete as Affected by Steam Curing Temperatures—Joseph J. Shideler and Wilbur H. Chamberlin

A Shortcut for Determining Reinforcement in Reinforced Concrete—V. Bogvad-Christensen

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Precast units offered savings in cost and maintenance of short-span bridges in Massachusetts.

Precast Units for Short-Span Bridges*

By ROBERT C. HANCKEL†

SYNOPSIS

Bridge replacement in Lowell, Mass., where minimum traffic interruption was necessary, led to the adoption of precast reinforced concrete units. Available mobile hoisting equipment limited the maximum weight of sections to about 12 tons which permitted small bridges to be precast as complete structures, while for larger bridges, subassemblies were precast and incorporated into the bridge by a cast-in-place deck. The precasting procedures and construction processes are described and illustrated.

INTRODUCTION

The City of Lowell, Mass., has about 56 bridges of one type or another which have been built through the years. This, of course, is an unusually large number of bridges for a city of its size and is due largely to the canal system which supplies water to the various industries. In view of past experience in replacing many of these bridges, Stephen Kearney, City Engineer for Lowell, came to the conclusion that precast units offered a considerable saving in initial cost and at the same time would provide bridges requiring very low upkeep. Experience has shown that it is much easier to get money for new projects than for maintenance work. Therefore, to keep maintenance costs low, it is very important to eliminate all such items as painting, etc., which, of course, is one of the advantages of good reinforced concrete.

The precast concrete units were made of conventional reinforced concrete. Prestressing was not attempted. The beams were not cambered as this was considered an unnecessary refinement that would complicate form work and, so far as we could judge, would give the structure no added strength.

*Presented at the ACI New England Regional Meeting, Boston, Mass., Nov. 11, 1949. Title No. 46-20 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 5, Jan. 1950, *Proceedings* V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than May 1, 1950. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Consulting Engineer, Boston, Mass.

Precast units for bridges may be classified as follows:

a. Those which make up a completed structure by themselves—except for grouting of joints, black top wearing surface and rails.

b. Those which make up only a part of the completed structure—the deck being poured in place to integrate the precast units into as nearly a monolithic whole as possible.

The decision as to which type to use depends a great deal on the weight it is possible to handle with ease. In the case of the Lowell bridges, 12 tons seemed to be the proper limit and this weight was just about reached in the case of the 52-ft Howard Street girders.

It is obvious that the first type (those which make up practically the completed structure) are—or should be—the most economical as they reduce field formwork to zero. This type is illustrated by the slabs for the Plain Street bridge, and has been used in other states in various forms, mostly for quite short spans of 20 ft or less. Again it must be emphasized that it is a question of what equipment is available for transporting and hoisting the units and that the hoist owner must understand fully what he is up against in placing the units; remember that the capacity of a mobile hoisting rig is greatly reduced as the boom is lowered towards a horizontal position. It is, of course, possible to place long precast units by other means than mobile hoisting rigs, such as cable ways, temporary monorails, etc., but the mobile rig is obviously the speediest method of placing units. The 18 slabs for the Plain Street bridge, for example, were placed by a mobile rig in 4½ hours, using four men in addition to the hoisting rig crew.

PLAIN STREET BRIDGE

The Plain Street bridge (Fig. 1) was built with precast units which made up practically the completed structure. The field work consisted

Fig. 1—Plain Street bridge, new structure

of: (1) placing the slabs on the abutments and piers, (2) filling in the dry mortar keys and installing the dogs and (3) placing the topping and erecting the rails—all of which was done quickly.

The position of the existing piers demanded different lengths of slabs, which would not be the case for new work where greater duplication of units would be possible.

The ribbed type of roadway slab was chosen to obtain the greatest depth for the minimum weight.

The design of the roadway slabs was based on the theory that a pair of ribs act as a unit. That is, the stringers were considered to be T-beams with a 6-in. x 5-ft flange, a 24-in. web and 16 in. total depth. It is possible to do this by means of a proper shear key (Fig. 2) and a device for keeping the ribs in close contact. In this case it was accomplished by the wedge-shaped key and U-shaped dowels, often called dogs (Fig. 3). Other schemes could have been used but the one shown seemed simple and economical.

The design of the 6-in. slab was based on the usual procedure prescribed by the American Association of State Highway Official's Specifications for main reinforcing perpendicular to the direction of traffic.

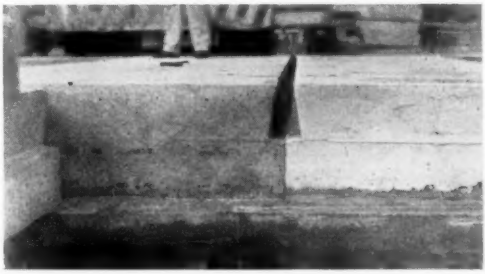

Fig. 2—Shear key used in the Plain Street bridge

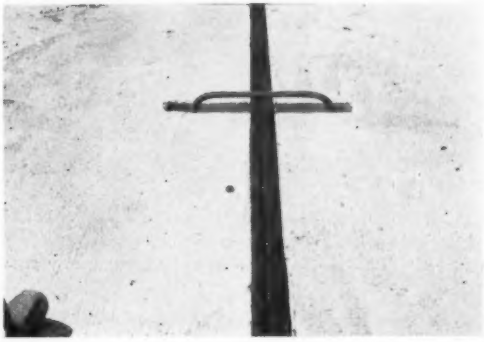

Fig. 3—Shear key and dog. The dog is held up; the final position is in the slot below the roadway surface.

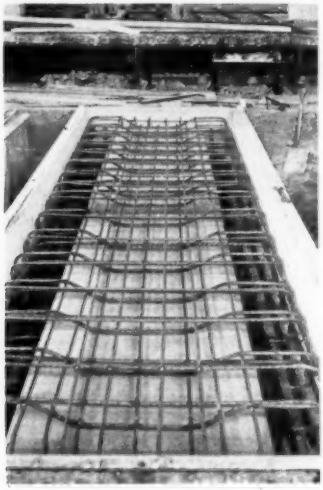

Fig. 4—Reinforcement in place in the roadway slab forms for the Plain Street bridge

Some slight economy of steel might have been made since the slab was fully supported on four sides, but this was ignored for the sake of uniformity in the spacing of bars. Because the slab was *not* continuous over more than two supports but at the same time was monolithic with the ribs, the design of this slab was based on the assumption of free supports in so far as positive steel was concerned, and as continuous for negative steel, the clear span being used in both cases. Fifty percent of the main steel was used in longitudinal distribution. A few top bars were added to take care of the negative moment over the cross rib. Fig. 4 shows the reinforcing in place in the slab forms.

As already mentioned, the design of the longitudinal ribs was based on two ribs working as a unit. For 5-ft spacing, *the pair* of ribs was required to carry 100 percent of one wheel load of 16 kips in accordance with specifications for an H-20 loading. This called for three 1-in. square bars per rib, one of which was terminated short of the supports and the other two made full length and hooked. The depth of the ribs was such that stirrups were not required for diagonal tension, thereby keeping everything as simple as possible for forming and placing steel.

The sidewalk slabs were made solid to simplify formwork and at the same time keep the total weight under 6 tons. The major portion of the required longitudinal steel was placed on the outside third to stiffen the free edge. The sidewalk slabs were anchored to the road slabs by full-length keyways (Fig. 5). This has not proved too satisfactory in prac-

tice. Possibly the sidewalk slab section should be changed to that of a ribbed slab similar to the roadway slabs and tied to them in some manner. To anchor the railing posts to the sidewalk slabs, 6 x 6 x ½-in. steel plates were placed flush with the top of the slabs and properly anchored. These plates provided an easy method for field welding the railing posts and has worked out very well in practice since ample tolerances are available, field drilling is eliminated and a very rigid anchor is obtained. The same detail was used for all bridges with excellent results.

All things considered, this method of constructing a bridge seems to offer some real economies for conditions similar to those of the Plain Street bridge. By narrowing the slabs and increasing the ribs, spans can be increased to approximately 20 or 22 ft with little increase in weight. For instance, by decreasing the width from 5 ft to 4 ft and reducing the thickness of the slab by ½ in. it is possible to use the *same depth of rib* and the same reinforcing as for the Plain Street bridge slabs for a span of 20 ft and H-20 loading.

HOWARD STREET BRIDGE

The Howard Street bridge is typical of the second type of precast

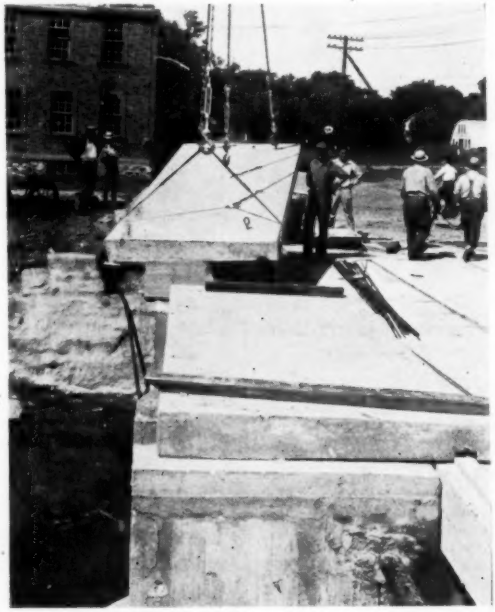

Fig. 5—Hoisting the sidewalk slabs for the Plain Street bridge. Note the full length keyways on the underside of the slab used to anchor the sidewalk slabs to the beams and roadway.

unit—that is, units that make up only part of the completed structure. The roadway and sidewalk slabs, together with the diaphragms, were poured in place. The advantage of this type is found in the elimination of all shoring and, as for any good concrete structure, the maintenance is low.

The shape of these beams was adopted for two reasons: (1) it was an easy shape to form and cast and (2) it provided a convenient shape for the field work. The wide portion at the bottom provided lateral stiffness, greater width for coverage of bars and a shelf for supporting the formwork for the roadway slabs. We called for a 1⅝-in. shelf as this is the finished dimension of 2-in. nominal thickness lumber. (The same held true for the dimension of 2⅝ in. at the top of the beam.) This was advantageous for formwork. Various views of the beams during construction and handling are shown in Fig. 6 and 7.

In the *design* of these girders, two loading conditions were considered: first, the girder was designed as a rectangular beam to carry the dead

Fig. 6—Reinforcing for the Howard Street bridge beams. All cages were welded and hoisted into place in the forms. Welding reinforcement saves time and insures the proper spacing of bars.

Fig. 7—Unloading the 52-ft beams, weighing 12 tons, seemed to tax the capacity of the hoisting rig more than any other job due to the lack of space which necessitated a low boom.

weight of the formwork and green concrete for the roadway slabs in addition to its own weight. The second step considered the design of the T-beam of which the precast unit is made an integral part. This T-beam, of course, carried all live load plus the load of the topping. The shear keys and the dowels may be likened to the intermittent welding of a built-up steel girder, the calculations following similar lines. The stirrups were used to anchor the slab against any uplift which might occur and they stiffened the beam and resisted diagonal tension. They also served as chairs for the spacer bars.

The same forms were used for all beams with little change. The forms were made long enough for the 52-ft beams and adjusted for the shorter spans. The forms were stripped in 24 hours. A ready-mixed concrete with a 4½ in. slump was spaded and vibrated after being placed in the forms.

The design of the precast units is simple and straightforward; but that is only part of the picture. The designer must give thought to integrating the units, using the poured-in-place slabs and diaphragms to accomplish this. The function of the roadway slabs is, of course, obvious. The diaphragms serve a double purpose in staying the beams and helping to distribute loads. The end diaphragms are all-important in uniting the beams into one common mass.

The A.A.S.H.O. Specifications (1944) state that "For T-beam spans over 40 ft in length diaphragms or spreaders shall be placed between the beams at the middle or at the third points." It seems to leave to the designer the question of thickness and maximum spacing. Even though most of our bridges are under 40 ft we have felt it wise to use at least one set of diaphragms. In this case we used two sets. There seems to be no criterion for the maximum spacing of these diaphragms; we suggest a spacing of 14 to 16 ft as this is close to the theoretical distance between truck axles. (Don't ask us to substantiate that idea with computations—it just seems like a good rule.)

The end diaphragms in the Howard Street bridge permitted generous tolerances in the length and positioning of the beams. The wise contractor will space the beams to a close tolerance to permit the use of prefabricated forms but he should be allowed some leeway in spotting the beams lengthwise. Actually, it is surprising how closely these beams can be placed to an exact position despite their considerable weight, 12 tons in this case. The beams as placed for final position are shown in Fig. 8.

DUTTON STREET BRIDGE

The Dutton Street bridge was similar in all respects to the Howard Street bridge except that traffic had to be maintained during construction. The old structure is shown in Fig. 9a and 9b.

Fig. 8—Girders in place and ready for formwork in the Howard Street bridge

Fig. 9a and 9b—Dutton Street bridge, old structure. Looks like it's good for H-1 loading (one-horse).

Contrary to the practice of putting construction joints at midspan of slabs, it was possible, here, to place a joint at the centerline of a beam; this was done in the field at the center-line-beam.

Traffic was maintained on the old bridge while one-half of the new bridge was constructed. All reinforcement was welded (Fig. 10) and the slab rods were extended beyond the construction joint for lapping with the second section of the bridge. High-early cement was used for the roadway and sidewalk slabs. Whereas we implied that the traffic had to be maintained on this bridge at all times, that was not exactly the case as it was necessary to keep traffic off the entire bridge for three days after the second half of the bridge slab was placed. This was done to stop the working of the beam adjacent to the green concrete until the *new* concrete had gained sufficient strength so that such working would not be harmful.

It was extremely gratifying to see how well the field operations proceeded in constructing this bridge which was the first of the bridges to be erected.

CABOT STREET BRIDGE

The Cabot Street bridge (Fig. 11) had three spans of approximately 35 ft each. This job is typical of what can be done to build a *continuous* structure using precast units. The precast units, of course, were designed in the same way as those for simple spans in so far as the dead load was concerned. In considering the live load, however, the beams were designed for continuity and therefore top steel was added in the slab over the piers as shown by the typical section through these piers in Fig. 12. To integrate the beams thoroughly the end detail shown in Fig. 13 was devised. It is obvious that it was essential to obtain as uniform bearing at the piers as possible, and we therefore increased the width of the diaphragms at these piers to nearly that of the piers them-

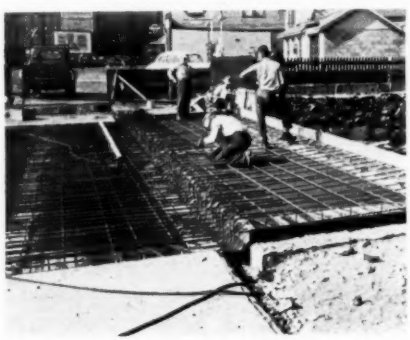

Fig. 10—Placing slab reinforcement for the Dutton Street bridge. Note the welder spot welding the bars. Welding proved to be very economical. A welder must know how much is needed—some are inclined to weld too much.

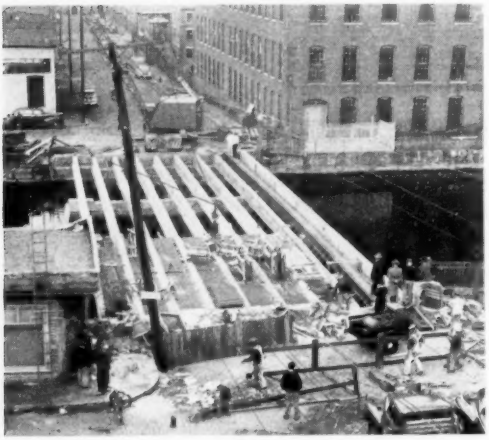

Fig. 11—Beams in place and part of the slab formwork for the Cabot Street bridge

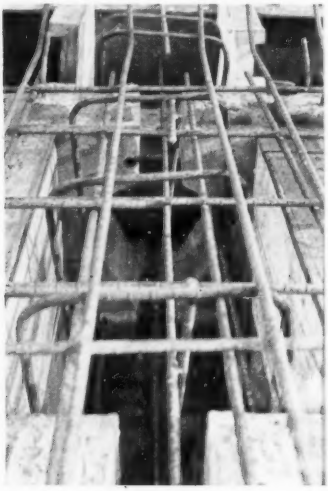

Fig. 12—Ends of beams at the piers, Cabot Street bridge

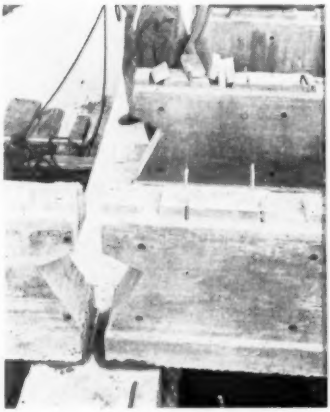

Fig. 13—Ends of beams at the piers, Cabot Street bridge

selves, and obtained continuity of the diaphragms in the manner shown in Fig. 13. The concrete and bars were *continuous* across the pier. Aside from the condition of continuity, the essential features of this bridge were similar to those for the Howard Street bridge.

CONCLUSIONS

Now that the bridges are built, how do we like them and what would we do to improve the design?

So far as the Plain Street bridge slabs are concerned, everything clicked very well except for the sidewalk slab anchoring scheme. There was some trouble with the lifting inserts. These inserts should be made with a large factor of safety. A rig *could* be devised to lift the slabs by clamping on to the *shear* keys. However, a simple looped rod projecting above the slab would seem to be the easiest way out if the *fabricating* and *stacking* of the slabs would permit their use; in this case it was planned to fabricate by pouring one slab on top of another and therefore the loop device was not permissible. Also the stacking prohibited their use.

In casting slabs of the type used for the Plain Street bridge, close casting tolerances are in order. We understand, that for the size slabs built, $\frac{1}{8}$ in. tolerances can be obtained easily with careful workmanship.

The precast beam jobs seemed to go smoothly as a whole. (In placing the first beam there was some trouble due to the rig not being adequate; but once an adequate rig was used, the beams were placed in jig-time. It took only a matter of $1\frac{1}{2}$ hours to place the first six beams for the Dutton Street bridge.)

The Howard Street bridge beams, due to their 12 ton weight, were a bit more difficult to handle. However, the contractors were resourceful and by the use of an additional smaller rig they were able to place these beams in their correct position with little trouble.

We believe that greater width at the bottom of the beams might be good, say 20 in. instead of 15 in., thus obtaining additional stiffness for handling purposes.

Regarding all the precast units, it seems to us they could be safely designed by the ACI Code rather than by the A.A.S.H.O. Specifications. It is not clear to us why A.A.S.H.O. stresses should be so much lower than those deemed safe by ACI. (Probably they are based on the supposition that bridges are sometimes built in places where concrete control may be questionable.) However, for the close control of concrete quality given these bridges, it would seem that the higher-stresses allowed by ACI would be in order especially since A.A.S.H.O. Specifications have not as yet gone into the question of precasting.

We believe that the recent experience gained on these jobs points to the possibility of using precast units to good advantage for short-span bridges. This is substantiated by the fact that the contractor employed only two carpenters and two laborers to erect all of the bridges —assisted, of course, by the hoisting rigs and a welder for the reinforcing steel.

The lesson to be learned from these jobs is, perhaps, that the simple way is frequently the most economical. We have been hearing a good deal about prestressing but we hold that such methods of building bridges are only economical in this country for long spans and where perfect control of all operations is possible. For the short spans, it seems better to use more materials and save the labor and the close control required for prestressing the steel to a known intensity.

PERSONNEL

The author, acting as engineer for the contractor, correlated the basic information regarding spans and sizes as given by the City Engineers' plans into working drawings, and submitted computations for the work in accordance with the stipulations in the specifications.

The Thompson and Lichtner Co. were consultants and inspectors for the work. The precast units were made by the Durastone Co. and fabricated at their plant in Saylesville, R. I. The W. J. Hurely Construction Co. of Lowell were the contractors. All work was under the supervision of Stephen Kearney, City Engineer, and his staff.

Only rarely, it seems, is the material on our library shelves summarized to present the progress made and chart the problems ahead.

Influence of Subgrades and Bases on Design of Rigid Pavements*

By KENNETH B. WOODS†

SYNOPSIS

This paper has been developed from research data, published reports, and experiences gained by observing the performance of both rigid and flexible pavements—particularly as related to subgrade soil textures and the use of base courses. Structural failures of rigid pavements, caused by large concentrations of exceptionally heavy loads, indicate the need for an evaluation of subgrades and bases in determining the most economical design of rigid slabs.

Indications are that the structural capacity of rigid pavements can be improved by the use of location procedures which utilize the best in topographic position and subgrade soil textures. For inferior situations —in regard to position and soils—the use of base courses must be evaluated against the economy of using slabs of increased thickness, more reinforcing steel, or combinations of the two.

It is concluded that it is not entirely feasible, with the present state of knowledge, to standardize rigid pavement design. Rather, the available data indicate that design practices should be developed by regions in which the subgrade soil, availability and type of base course materials, climatic conditions, and traffic volumes and loads are evaluated.

INTRODUCTION

Early highway designers recognized that soils vary in their capacity to support loads, and many data are available on such problems as frost heave, embankment settlements and foundation failures in peat bog and muck areas. However, specific information on the strength characteristics of subgrade soils in relation to design requirements for rigid pavements is not so abundantly available.

It is the purpose of this paper to discuss the importance of subgrades to the performance of heavily-traveled rigid pavements, and to con-

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†Member American Concrete Institute, Civil Engineering Dept., Purdue Univ., West Lafayette, Ind.

sider the use of base courses and subgrade treatments for improving the structural capacity of these pavements. The importance of considering rigid pavement design in the light of the availability and the characteristics of constructional materials, the prevailing soil texture and the climatic variable, is emphasized.

Subgrade

The correlation between rigid pavement performance and the texture of the underlying subgrade soil has been difficult to determine, probably because of the over-design of rigid slabs in relation to the loads carried during the 1920's and much of the 1930's. However, high concentrations of heavy loads and the relatively large number of overloads occurring on many of the primary roads, both during and following the second World War, have caused many structural failures. Study of these failures, diagnosed early as "pavement pumping," has made possible the correlation between pavement performance and subgrade soil textures. Also, the experiences gained by engineers working with military airfield runways carrying exceptionally heavy wheel loads have added considerably to the store of knowledge.

Base courses

Base courses have been used under rigid pavements in the New England states and to some extent in Michigan, Wisconsin, and Minnesota for a great many years, primarily because of the frost heave problem. In the Middle West, the use of bases was largely on an experimental basis through the 1920's and 1930's, but the use has become standard in many areas where heavy concentration of traffic is anticipated and where the location is in areas of clay-like materials. Although granular materials, *i.e.*, sand, gravel, crushed rock, granulated slag and others, have been most used as base courses, both field and laboratory experiments have shown the feasibility of using soil or semi-granular materials with admixtures—particularly where granular materials are not economically available.

SUBGRADE INFLUENCE

All soils originate from rock materials of widely varying characteristics, ranging from sedimentary rocks of shale, sandstone and limestone textures, through the basalts and lavas of igneous origin, to crystalline rocks produced by metamorphic processes. In many situations, soils developed from rocks in place have been transported by continental and mountain glaciers, wind, or water. Soils are modified further by frost action, percolating water, the growth and decay of vegetation and by bacteria and other organisms. These variables—character of parent materials, method of transportation, climate, plant growth, length of

time of weathering and topographic position—combine to produce soils with a wide range of textural, physical and chemical characteristics, as well as variations in the characteristics of the various layers or horizons of the soil profile. The variations in soil characteristics complicate the procedure of pavement design to a marked degree.

Soil classification

Means of identifying and classifying various types of soils have been developed utilizing visual appearance and physical and chemical tests. The color is an obvious feature which can be recorded, and the structural appearance of the soil is frequently noted. Early attempts to classify soils for engineering purposes by tests values were made by Terzaghi,[1] Hogentogler, Terzaghi, and Wintermyer[2] and by Eno.[80] Efforts to record soils information with respect to highway location, design and construction include the work in New Hampshire[3] and soil survey practices in Ohio.[4] A summary of soil survey practices in the United States was developed by Muir and Hughes[5] in 1939.

Much effort has been expended in developing means of classifying soils, including the research done by the Bureau of Public Roads. This grouping was revised in 1946 by a Highway Research Board committee on classification of materials for subgrades and granular type roads.[7] A different approach was used by Casagrande for recording and classifying soil characteristics.[6] An adaptation of the pedological system of soil classification for engineering usage has been applied with success in Michigan,[8] as well as in several other states including Missouri, North Carolina, Indiana[96] and others.

Subgrade soil textures

The actual evaluation of various types of subgrade soils for their structural capacity, as related to the design of rigid slabs, has not been completed. Early attempts to evaluate the supporting power of subgrade materials include the Bates test road,[13] the Arlington tests[47] and others. Hogentogler and Terzaghi,[28] in 1929, stated that "The support furnished by different subgrade soils varies widely with regard to intensity of support, character of support and uniformity of support, and thus permits a differentiation of considerable amount in pavement requirements."

In 1935, Janda[29] showed an outstanding correlation between the performance of a rigid pavement and the character of the subgrade soil. A performance survey of pavements showed uniformly bad performance in one type of soil as indicated on an agricultural soil map and exceptionally good performance on soil of different character. Woods and Gregg[31] have shown that a relationship exists between the performance of pavements and the character of the subgrade soil. An Indiana per-

formance survey has shown, further, that practically all of the exceptionally old rigid pavements, *i.e.*, those older than 20 or 25 years, are to be found on good soil materials. It is also significant that the subgrade-traffic factor was of greater importance than were minor differences in pavement design, *i.e.*, thickened middle versus thickened edge, etc. In a later report, Sweet and Woods[30] show that mapcracking and related disintegration in concrete pavements constructed with inferior coarse aggregates is severe on pavements located on clay-like materials, but that this problem is relatively minor when pavements are located on well drained sands and gravels.

Warping of concrete pavements has been given consideration by a committee of the Highway Research Board. Allen[34] has shown that the problem is related to the climate-soil variable. The 1946 report of the committee states that "Warping is caused by swell or shrinkage of the subgrade soils" and the problem "is definitely associated with the characteristics of the subgrade soils." Expansive soils and plastic clay-like materials were the chief offenders.

The construction of embankments over peat bogs has been one of the more difficult soil problems,[25] and many ingenious methods for solving the problem have been developed, including blasting, the use of sand wells[26,27] and the so-called "floating" method.

The position factor

The position of the pavement with respect to the horizons of the soil profile is important. This point is well illustrated by an analysis of pavement pumping on primary roads constructed in regions covered by materials of the Illinois glacial period, *i.e.*, Southern Ohio, parts of Southern Indiana and Southern Illinois. These soil materials consist of 18 to 20 in. of a "rock-flour silt" on relatively deep deposits (in some cases as deep as 10 ft) of a dense, plastic clay-pan soil. The topography in areas covered by Illinoian drift is generally monotonously flat, being broken only by streams and related erosional features. On heavily-traveled roads, constructed with no subgrade treatment or base course, pumping will invariably occur in the cut sections of the valley walls of the streams and tributaries. Here, the clay-pan material forms the subgrade soil. In contrast, pumping is of minor consequence in the upland flat areas where the subgrade consists predominantly of soils of silt size. This is true despite the fact that these silts are frequently water-logged because of a perched water table. Thus, a relationship exists between pavement performance and the position of the pavement with respect to the B (clayey) horizon of the soil profile.

Influence of position in pavement performance is illustrated further in connection with frost damage. Frost heaves have been known to

occur in areas of flat-lying lake sands under conditions of high water table. In contrast, difficulties are uncommon in areas of dunes composed essentially of the same textures as the adjacent low-lying sands.

Frost action

The effect of frost action on rigid pavement performance has been studied at considerable length, and both laboratory and field studies have resulted in routine design practices which minimize the detrimental effects of frost. The work of Taber[15] is still a classic on the laboratory evaluation of frost heaving, while Beskow[16] has made intensive studies in Europe. Winn[17] and Slesser[18] have studied the use of admixtures to minimize the effects of frost action in both base courses and in subgrades. The problem is also of major importance in connection with the design and construction of runways.[19]

Pavement pumping

The problem of pavement pumping was practically non-existent throughout the 1920's and 1930's when axle loads rarely exceeded 18,000 to 20,000 lb; but, since about 1940, there have been wide-scale structural failure of rigid pavements located on the primary highway systems. The problem was first described by Poulter, and a committee of the Highway Research Board on maintenance of joints in concrete pavements as related to the pumping action of slabs was established in 1943.[39,42] Woods and Shelburne[38] reported that the problem could be correlated in general with the texture of soils, showing that clay-like materials were particularly bad. In a later report from Indiana,[45] a correlation was established between pavement pumping and the occurrence of overloads on the primary roads of the state system. Whitton,[36] in a Highway Research Board committee report, indicated the relationship between structural failures and heavy axle loads, while Lynch[37,40] reported a decided increase in the frequency of heavy axle loads. More detailed information was reported by Dimmick[41] in 1945. Boughton[43] summarizes the existing information as to how the truck size and weight limits affect highway design.

Subgrade modulus

Because of the peculiar character of the problem of pavement pumping, it has been difficult to analyze the stresses in slabs and to develop theoretical designs for the solution of the problem. The situation is complicated greatly by the fact that, at the present time, there is no way of knowing what to expect in regard to future axle loads. The Westergaard analysis[46] is difficult to apply because of the uncertainty of the subgrade support and the magnitude of loads. Kelley,[48,49] in 1939, discussed the modulus of subgrade reaction in relation to the computed stresses in paving slabs. Since pumping consists of the actual extrusion

of a portion of the subgrade support, it is not likely that a modulus of subgrade reaction value can be of use in such cases. Teller and Suthland[47] recognized the importance of the climatic variable in 1945 when they stated that "The conditions of pavement support are probably more uncertain during the period immediately following a thawing of the subgrade than at any other time." The design of airport runways for heavy loads[50,52,53,54] shows the trend of modifying the modulus of subgrade reaction concept.

PAVEMENT DESIGN AND SUBGRADE TREATMENT

Engineers are compelled to evaluate rigid pavement design principles in the light of the large number of overloads which the primary systems are carrying, as well as numerous repetitive loads which are near the legal weight limit.

Reinforcement

Continuous reinforced pavement is a subject of recent interest and several states have constructed experimental sections, including Indiana,[56] Illinois[57] and New Jersey.[55] This design feature is particularly attractive where the soils are exceptionally poor, where granular materials for subgrade treatments are difficult or expensive to obtain, and where high concentrations of heavy industrial travel are anticipated.

Increased thickness

In airport construction, increased thicknesses of runways and taxiways have been tried. At Patterson Field, 21-in. thick concrete runway slabs and 25-in. taxiways were constructed to carry gross plane loads of 300,000 lb on two tires.[53] These designs were obtained through the use of full-scale load tests made at both Wright Field and at Lockbourne.[52]

Soil compaction

Procedures for improving the foundation, embankment, and subgrade conditions have also received special attention. Particularly important is the use of specifications and compaction control for subgrades and embankments.[58,59,60] Attention has been given to such features as optimum depth of layer, optimum moisture content of the soil being compacted and type and weight of roller for compacting various textures of soil.[61] The use of heavy equipment was given emphasis during the last World War, particularly in connection with the compaction of bases for airport runways carrying heavy wheel loads.[62]

Compaction of cohesionless materials has been the subject of considerable research for the past 20 or 25 years, and with the increased use of granular base courses, the subject takes on added interest. The use of vibration[64,66] and the use of explosives[65] have received a great deal of attention in recent years. The solution to the problem is not entirely

available, however, even today. This problem is also of major importance in the construction of flexible pavements.[68]

Drainage

One obvious method of improving subgrade support in many situations is through drainage. In early designs, the high level profile was often employed in areas of extremely flat topography to secure good subgrade drainage and to minimize the accumulation of snow drifts in the winter. With the advent of wide berms and gentle ditch slopes, subsurface drainage has become a matter of extreme importance. Interest has increased in soil moisture content,[75] as well as in the study of moisture movement in soil.[69,70,72] Much attention has been given to the removal of water where pavement pumping has become a problem.[71] Also, the gradation of back-fill materials has been studied and evaluated in an attempt to minimize or eliminate the infiltration of soils in the drains.[73,74]

BASE COURSES

In the New England states and in the northern tier of the Middle Western states and Canada, much engineering experience has been collected in the use of deep base courses of granular materials, *i.e.*, sand, gravel, crushed stone, cinders, granulated slag and slag. The primary function of these base courses has been to minimize or eliminate frost heave. Also, excellent data and much experience have been collected in the use of sand clays as surface courses and as base courses for bituminous pavements in the Atlantic and Gulf Coastal Plain states. However, these experiences have been directed more toward low-cost road surfaces and base courses for low-cost pavements than for base courses for rigid pavements.[83,90]

Ohio made early use of granular base courses under rigid pavements to effect better drainage in cut sections through clay-like materials and to improve the supporting power of the subgrade material.[86] Missouri[87] has conducted extensive field experiments with bituminous materials and other admixtures in an effort to prevent or retard water movement through subgrade soil.

A wide variety of materials have been employed, at least on an experimental basis, for subgrade treatments. Eno[80] used granulated slag, soil cement, lime, stone and slag screenings, crushed stone, crushed slag, sands, gravels and other materials in field experiments to evaluate their use for improving subgrade capacity. Henderson and Spencer[82] reported on the performance of a heavily-traveled road in which eight different types of subgrade treatment were employed. Included were dune sand, limestone dust, limestone screenings, three sections of bituminous-soil mixes and a water-saturated section. Several of these ex-

perimental treatments showed outstanding improvement in the performance of heavily-traveled U. S. No. 30 in northern Indiana which was pumping badly in adjacent untreated sections. Massachusetts has used gravel and stone base courses under rigid pavements since 1914 in an effort to facilitate drainage and to eliminate frost heaves.[78] Coburn, in the same article, estimates the cost of 12 in. of gravel base to be approximately equal to one in. of paving concrete.

The use of tar as a stabilizing agent has been given attention.[92] Large-scale installations of soil-cement mixtures have been used throughout the United States largely for base construction of low-cost roads, although occasionally as bases for rigid pavements.[85,94] A great amount of research data are available on the use of asphaltic materials for subgrade treatment,[84] and a large mileage of bituminous-treated base courses has been constructed throughout the country, particularly for low-cost road construction.

Granular bases without admixtures have been employed with increasing favor, particularly in regions where there is an abundant supply of sand, gravel, stone, etc. The structural characteristics of various granular mixes has been studied.[77,81,88] However, the use of "pervious" versus "impervious" base courses is still a subject for debate. Deklotz[79] has studied the effect of gradation on the stability of granular materials, and Yoder and Woods[93] reported on the optimum "soil" content of certain granular materials from the standpoint of compaction and strength characteristics.

Philippe has reported[54] unsatisfactory performance of nonreinforced concrete test slabs tested to failure on granular subgrades in comparison with the performance of similar slabs constructed on adjacent untreated raw soil subgrades. He concludes that ". . . the use of a base course should be discouraged for airfield pavement." Masheter[86] reported a somewhat similar experience with nonreinforced highway slabs constructed during the war without load transfer devices and without reinforcing steel. Indiana has had at least one similar experience in connection with a nonreinforced paving project constructed during the war. Woods, Green and Sweet[45] reported the faulting of slabs at both joints and cracks of almost 50 miles of heavily-traveled highway pavements constructed on deep sand subgrades. The indications are that consolidation of the sand takes place under vibration caused by impact and the passage of heavy loads. It follows that caution must be exercised in the use of granular materials for subgrade treatment for both rigid and flexible pavements, from the standpoint, at least, of compaction.

DISTRIBUTION OF SOILS

In discussing the influence of subgrades and bases on the design re-

quirements of rigid pavements, it is pertinent to consider the distribution of soils, their characteristics and problems and the availability of constructional materials. For the purposes of this discussion, it is reasonably appropriate to consider these features on the basis of soil origin, *i.e.*, water-deposited, wind-deposited, ice-deposited and soils developed from bedrocks in place.

Fig. 1 shows, in a general way, the distribution of the surface materials of the continental United States. Fig. 2 shows the major physiographic bounderies adopted by Fenneman.[99]

Ice-deposited materials

Glacial debris covers all of New England, most of New York, a large portion of the Middle West, and is to be found in the northern tier of states in the Great Plains and in the West. The character of the glacial deposits varies considerably from section to section, depending on several variables including the texture and topography of the rocks over which the glacier passes and depth of ice sheet. However, for purposes of pavement design and evaluation, glacial deposits can be discussed under four general headings: (1) New England and part of New York, (2) Middle West, (3) Northern Great Plains and (4) Rocky Mountains and Far West.

The New England drift is generally shallow and is of semi-granular texture. The drift in this region is derived largely from weather-resistant metamorphic and igneous rocks; therefore, deep beds of plastic materials are exceptional. The character of the drift, plus its general shallowness, gives rise to problems of drainage and frost heave, particularly where inter-bedded silts are encountered. Materials of construction, including granular materials for base courses, are, in general, abundantly available either from kames, eskers, outwashs and terraces, or from underlying bedrocks containing high-grade materials of both igneous and metamorphic origin.

The Middle West drift, in contrast to that occurring in the New England states, is deep and fairly plastic. The ice sheet in this region passed over sedimentary rocks including limestones, shales, and sandstones, which accounts in part for the character of the drift. Constructional materials are generally abundant, there being a liberal supply of both sands and gravels and underlying limestones of good quality.

The drift which covers the Northern Plains area is generally of a silty texture. Sands and gravels are found to only a limited extent, and the bedrocks are of poor quality. It follows that constructional materials and granular materials for base courses are frequently difficult to obtain. The drift materials of the Rocky Mountains and states of the

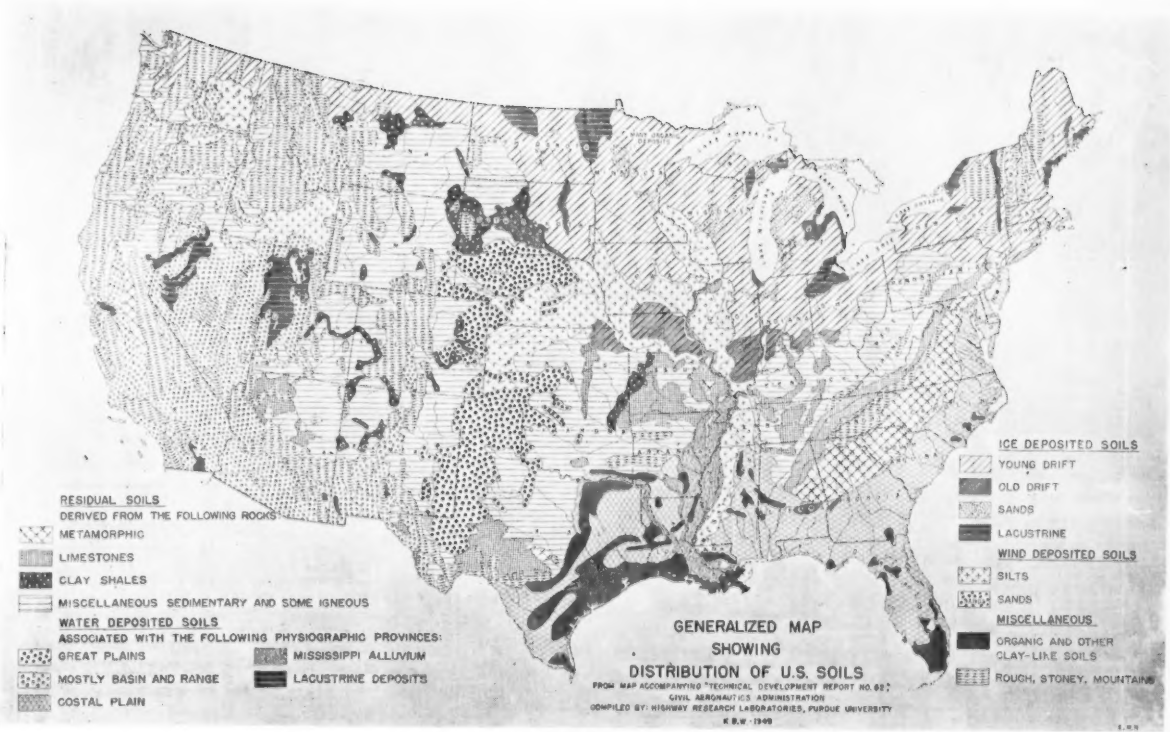
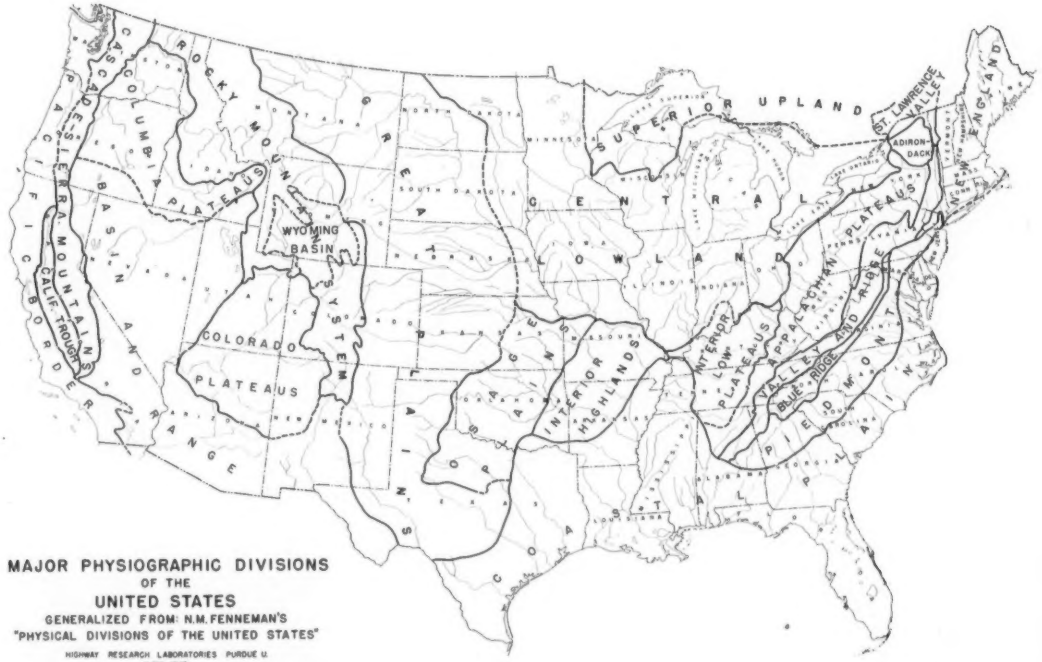

RESIDUAL SOILS
DERIVED FROM THE FOLLOWING ROCKS
METAMORPHIC
LIMESTONES
CLAY SHALES
MISCELLANEOUS SEDIMENTARY AND SOME IGNEOUS
WATER DEPOSITED SOILS
ASSOCIATED WITH THE FOLLOWING PHYSIOGRAPHIC PROVINCES:
GREAT PLAINS
MOSTLY BASIN AND RANGE
COSTAL PLAIN
MISSISSIPPI ALLUVIUM
LACUSTRINE DEPOSITS
ICE DEPOSITED SOILS
YOUNG DRIFT
OLD DRIFT
SANDS
LACUSTRINE
WIND DEPOSITED SOILS
SILTS
SANDS
MISCELLANEOUS
ORGANIC AND OTHER CLAY-LIKE SOILS
ROUGH, STONEY, MOUNTAINS
GENERALIZED MAP
SHOWING
DISTRIBUTION OF U.S. SOILS
FROM MAP ACCOMPANYING "TECHNICAL DEVELOPMENT REPORT NO. 69"
CIVIL AERONAUTICS ADMINISTRATION
COMPILED BY: HIGHWAY RESEARCH LABORATORIES, PURDUE UNIVERSITY
K.B.W. 1949

Fig. 1



MAJOR PHYSIOGRAPHIC DIVISIONS
OF THE
UNITED STATES
GENERALIZED FROM N.M. FENNEMAN'S
"PHYSICAL DIVISIONS OF THE UNITED STATES"
HIGHWAY RESEARCH LABORATORIES PURDUE U.
K.B.W. 1949

Fig. 2

Northwest are generally of a granular texture. Constructional materials are abundantly available.

Water-deposited materials

Water-deposited materials occur in many sections of the continental United States. Important areas include the Atlantic and Gulf Coastal Plain, large areas of the Great Plains, and the Basin and Range province of the Far West.

The Coastal Plain region consists mostly of sands, sand clays, and some clays deposited under marine or semi-marine conditions. Extensive deposits of clay are to be found, particularly in Texas, Alabama and Mississippi, and vast deposits of Mississippi alluvium occur primarily in Louisiana. Good quality constructional materials, other than sands and sand clays, are at a premium.

The surface deposits of much of the Great Plains consist of a deep mantle of material presumably deposited as outwash derived largely from the Rocky Mountains to the west. Gravels are found in the plains in close proximity to the mountains in terraces of the larger streams, such as the Platte River. The textures of the subgrade soils are in general sandy or silty; and this feature, combined with rather low rainfall, presents a situation whereby good subgrade support is offered for all types of paving construction. Sands are abundantly available for use as base courses, but other good quality constructional materials are generally unavailable.

The Basin and Range province offers little difficulty in regard to subgrade problems or constructional materials since the water-deposited sediments contained in the valley sections are generally granular and free-draining. Rainfall is, of course, very low in this region.

Wind-deposited materials

Wind-blown silts occur throughout the Mississippi and Missouri valleys; in the Great Plains, paricularly Kansas and Nebraska; and in the Northwestern states including certain sections of Washington and Idaho. These soils consist almost entirely of particles of uniform size. Materials of construction are practically nonexistent except in those areas where gravels or bedrock of satisfactory quality are covered only thinly with silt. Owing to certain conditions of structure, wind-blown silts are generally well drained. These materials offer moderately good subgrade support. Wind-deposited sands occur in many sections of the country with the dunes of the plains in Texas, Colorado and Nebraska being especially important.

Bedrock regions

Soils developed from bedrock in place are widespread in the continental United States. However, those of importance from the stand-

point of subgrade support and rigid-pavement design are found generally east of the Great Plains, lying between the Coastal Plain province on the south and east and the deposits of glacial drift on the north.

The Piedmont section, sandwiched between the Coastal Plain on the east and south and the Blue Ridge Mountains on the west, is the only large area which contains exposures of crystalline rocks from which deep residual soils have been developed. The soils in this area are plastic and troublesome. Constructional materials are often at a premium, although some gravels are found at or near the "fall line" hills and in terraces of the larger streams. High-grade quarry rock is sometimes available, particularly in some granite regions.

Soils developed from limestones and limestones and shales are found throughout the Ridge and Valley province, the Kentucky Blue Grass region, the Nashville Basin, and the Ozark Mountains. The soil materials in general are exceptionally plastic, and pavement pumping is a prevalent problem on most of the heavily-traveled roads. Constructional materials and material for base courses are generally available only from quarry rock, but occasionally from terraces of the larger streams.

Alternate beds of sandstones and shales are found throughout the Appalachian Plateau of Pennsylvania, West Virginia, eastern Kentucky and western Ohio, as well as in sections of Oklahoma, Kansas and Arkansas. Landslides, difficulties in embankment construction, lack of constructional materials and poor subgrade support are of major concern.

CONCLUSIONS

On the basis of both published information and the combined experiences of many highway engineers, several facts pertaining to the influence of subgrades and bases on the design of rigid pavements appear vital. Available data indicate that subgrade soils constitute a major variable from the standpoint of support offered to rigid pavements carrying exceptionally heavy loads. Although some of the major problems including frost heave and peat-bog subsidence have been solved, the problems of pavement pumping and subgrade strength evaluation remain without an exact solution.

On the basis of the distribution of soils and materials of construction, it seems logical to approach the problem of design with the regional-soil concept in mind. Where granular materials are abundantly available, the use of granular base courses in combination with slabs of moderate thickness, nominally reinforced, appears to be worthy of consideration. Where granular materials are at a premium, the use of soil stabilization employing bituminous materials or other admixtures and continuous reinforcement might well be the best answer. In sections where high concentrations of heavy loads are to be anticipated, it may be

necessary to use base courses and above-average amounts of reinforcing steel.

Drainage installation, the use of a high-level grade line and an evaluation of the position of the pavement with respect to the various layers of the soil profile can hardly be over-estimated. The matter of compaction of granular materials for base courses remains a problem of considerable magnitude, particularly when construction is attempted on plastic soils.

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Discussion of a paper by Kenneth B. Woods:

Influence of Subgrades and Bases on Design of Rigid Pavements*

By HENRY AARON, L. M. ARMS, T. J. KAUER and CHARLES W. ALLEN, L. A. PALMER, TILTON E. SHELBURNE and PHILLIP L. MELVILLE, and WILLIAM VAN BREEMEN

By HENRY AARON†

The author has emphasized the importance of the subgrade soil in relation to the design of rigid pavements. Its influence on pavement thickness requirements is probably greater than any other single factor. This is evident when one realizes that it is the subgrade soil that actually supports the loads imposed on pavements. The pavement, in addition to providing a surface capable of resisting the wear and tear of traffic, must serve to distribute the wheel load pressures to the subgrade in such a manner as to prevent failure in the subgrade. The more unstable the soil, the greater is the necessary thickness of the pavement. This resolves the problem into evaluating the stability of the soil and determining its relationship to pavement thickness and magnitude of the applied loads.

Owing to the variation in the stability of a given subgrade soil as influenced by such environmental factors as drainage, climate and the soil profile, the only satisfactory method of design presently used is based on a correlation between the subgrade soils and the performance of pavements under actual service conditions. In most cases soils are classified in accordance with certain physical properties and textures; and from observed behavior, pavement thickness requirements are estimated. The Bureau of Public Roads grouping and a modification of this as proposed by the Highway Research Board are such classifications.

For airport pavements the Civil Aeronautics Administration has adopted a soil grouping similar to that of the Bureau of Public Roads but has added a subgrade classification which takes into account the performance of the soils under variable conditions of drainage and climate. The subgrade class

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which governs the pavement thickness requirement is determined from a study of rainfall and frost records of the region, the topography of the site, and the arrangement of the different layers of the soil profile. This system, correlated with pavement performance, is proving very satisfactory.

The CAA is constantly checking pavement performance to define more accurately its pavement needs in relation to soils and traffic. Presently, the CAA is making a detailed pavement condition survey of selected airports throughout the country in an effort to bring design standards up to date. Pavement condition surveys have developed some of the most important information with respect to airport pavement design.

It has been definitely shown that taxiways, aprons and ends of runways are the most critical areas from the standpoint of aircraft use. Pavement failures have occurred on these areas while pavements of the same type and thickness on the runway proper showed no signs of distress. As a result the taxiways, aprons and runway ends are constructed to a greater thickness than the portion of the runway between the thickened ends. Also, these studies have proven that the impact, in a normal landing, may be entirely ignored in runway pavement design.

From the information obtained as a result of the correlation of subgrade soil behavior with pavement performance, it is believed that the physical properties of soils, as determined by the standard methods of tests, when combined with a proper interpretation of local conditions at a particular site, can serve as a basis for the development of standards for rigid pavement design which may be applicable in all parts of the country. This is especially true for airport pavements because of the relatively few aircraft types and the fact that the same types are operated in all regions. Also, for safety reasons, permissible aircarft loadings are nationally controlled.

The author suggests that the problem of design be approached with a regional-soil concept in mind. It is not clear in what manner this concept will be applied. It seems that in most instances regional or area concepts furnish general information but a detailed soil survey, including borings and soil tests, serve as a basis for the design of a particular project. In such a procedure the regional concept could supply the drainage, climatic and other environmental data for estimating the behavior of a soil having certain field characteristics as disclosed by the soil profile.

By L. M. ARMS*

The author has presented an excellent review of a great mass of data on the relationship of natural subgrades and base courses to the performance of concrete pavements which they support. In so doing he has digested some 99 papers listed in the bibliography, supplementing the data therein with his own wide observations of pavement performance.

It seems important to point out the need of good engineering judgment and a thorough study of the local conditions involved in each case when an

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attempt is made to correlate the reports of such widely varying research as that covered in the 99 papers. One does not have to search too closely to find support for diametrically opposite conclusions among the material presented. Careful study of the local conditions will usually give a good lead as to the reasons for the conflicting data although, at times, the failure to recognize the importance of significant factors may lead to their omission from the report. In that case it is extremely difficult for one not well acquainted with the areas involved to determine the reason for the conflicting conclusions.

Basically there can be no doubt of the significant influence of subgrade soils on concrete pavement performance, nor can one question the possibility of using subbases of selected materials, properly placed, to correct subgrade conditions that would lead to bad pavement performance. The important point for engineers to recognize is *when the subbases are necessary and when not necessary*. Like the doctor who knows the value and use of his remedies, he should know when not to use them as well as when to use them. Failure to recognize this may lead to increased cost with no corresponding gain in pavement performance.

Conditions that justify subbases

Observations of pavement performance under a wide range of conditions indicate there are three conditions which most commonly justify the use of subbases under concrete pavements. First, in areas of deep frost penetration the replacement of existing subgrade materials with free draining materials is one of the essential steps in the control of frost heaving. Second, in semiarid regions, soils having high swell characteristics require special treatment which may include subbases to prevent pavement distortion due to expansion of the soil when it absorbs water unequally during periodic rainy seasons. Third, the most common reason, using subbases on subgrades which would otherwise lead to pumping.

It is possible to identify soils and conditions which call for correction in each of these cases and the necessary procedures are fairly well understood. Additional data are needed as to the minimum depths of subbases which will prevent pumping and research to develop these data is currently in progress. In each case granular materials are commonly used but where suitable natural materials are not available, or are available only at a high cost, properties of the existing subgrade soil may be modified by additions of other materials, for example, cement to form a soil-cement or cement modified soil subgrade.

When subbases are used to correct either of the three conditions mentioned, there will be some improvement of the load supporting properties of the subgrade and this may be taken into consideration in the design of the pavement. There is need for better evaluation of the improvement of load carrying capacity of subgrades through use of granular subbases. Tests over a long period of years by both American and European engineers have shown low unit pressures, in psi, on subgrades under loaded concrete pavement. This is due to the large area over which loads are distributed. Since these

pressures are low, subgrades with high bearing value are not required structurally and improvement in bearing value of subgrades permits relatively small adjustments in rigid pavement thickness. Rarely, if ever, can the use of granular subbases be justified on the basis of improved bearing capacity. They should be used for the special purposes previously enumerated but their function in these cases should be understood and any tendency to extend their use into other fields should be resisted.

It would seem that the major value of the soils and physiographic maps which the author presented, and of similar local maps with their larger scale and corresponding greater detail, lie in their ability to differentiate between areas which require the most careful study by the soils engineer and other areas where a less thorough study will suffice. This permits the engineer to allocate his time and efforts to the best advantage. These maps will not enable the design engineer to do without the soils specialist; they will permit him to use the specialist more efficiently.

By T. J. KAUER and CHARLES W. ALLEN*

The author is to be complimented on his presentation of the development of our knowledge of the influence of subgrades and bases on design of rigid pavements, and an interesting discussion of the soils and highway construction materials of various regions of the United States. His bibliography is particularly comprehensive, containing references to nearly all of the important published data on the influence of subgrades on rigid pavement performance.

The writers would like to clarify one point which Woods has raised as to performance of concrete pavements on subbases in Ohio. Referring to a paper on "Design and Construction of Subbases," by P. E. Masheter of the Ohio Department of Highways, he states that Ohio's concrete pavements built without reinforcement or load transfer devices have experienced performance similar to that noted by R. R. Philippe at Lockbourne, *i.e.*, greater slab cracking on granular subbase than on adjacent untreated subgrade soil. In describing Ohio's experience with granular subbase material, Masheter did point out some cases in which performance of rigid pavements on subbases had not been entirely satisfactory. As Woods states, these experiences were associated with concrete built under war time restrictions without load transfer devices or reinforcing steel of any kind. These jobs had 20 ft contraction and 120 ft expansion joint spacing. Faulting of as much as ¼ to ½ in. has developed at many of the joints. However, there has been little pumping on these jobs and, as pointed out in Masheter's paper, over-all performance of these pavements on granular subbases has been much better than that of similar pavements built without subbases.

Regional soil concept

The "regional soil concept" described by the author has been recognized and applied to highway problems in Ohio for a number of years. The

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important effects of the geological and soil forming processes which have given rise to the subgrade soils and the materials of construction in this state are readily demonstrable. For example, considerable distress in the form of pavement pumping and faulting of concrete pavements is apparent on clays of the lake plains of northwestern Ohio, while generally excellent performance has been experienced with concrete pavements constructed on terraces composed of gravel and sand formed as glacial outwash deposits in a number of the major valleys of the southern part of the state.

Glacial gravels, deposited directly from the ice and those laid down as glacial outwash, constitute the source for a great deal of aggregate that goes into highway construction, both as subbases and as coarse and fine aggregate in pavements and structures.

In Ohio, as in many neighboring states, concrete pavements, in general during the thirties were constructed directly on soil subgrades using for the most part, a 9-7-9 thickened edge design. These pavements generally served well under the then prevailing traffic. However, with the increased loadings and frequency of heavy commerical vehicles which have developed since the early war years and continued to the present time, many of these older pavements have shown signs of considerable distress in the form of pumping; faulting and general breakage. As is to be expected, most severe failures developed on the heavy clay soils of the glacial lake plains in the northwestern part of the state and the predominantly silty clay soils of the till plains through the central and southwestern portions of the state. Relatively little distress was evidenced in pavements such as U. S. Route 20, east of Cleveland, lying on beach ridge sands through most of its length.

Granular subbases

During the late twenties and early thirties there was practically no systematic use of granular subbases in Ohio. However, a limited number of test sections, using various subbase treatments, were built under the supervision of F. H. Eno. The materials used ranged from graded aggregates to treatments with limes and oils; all of the treatments were in trench sections. Eno rated the performance of the various sections by means of crack surveys. These surveys showed that where granular materials were used there was more cracking than in the control sections without subbases. His explanation for this cracking was that it was probably due to additional compaction of the subbase material. Early in the life of these experimental sections Eno recognized the need for drainage of the porous granular subbase materials, a feature which had not been provided in his test sections.

With this background, when the use of subbases became general in this state, the need for compaction of the material was stressed and drainage provided. The first project on which a considerable amount of subbase material was used was constructed in the western part of the state in 1938. Incidentally, this pavement is still in good condition. From 1939 to about 1942 relatively few jobs with subbases were built. Since 1942 granular

subbases have been generally used under concrete pavements on all fine textured soils. Thickness of these subbases ranged from 12 in. to as much as 24 in., with 12 and 18 in. being the most common. However, the performance of some sections constructed on subbases only 6 in. thick indicated that the greater thicknesses previously used were unnecessary. Currently, the state is building most of the subbases 4 to 6 in. thick.

Over a period of years, since the first general use of granular subbase, Ohio has modified grading requirements from time to time in light of experience with various materials used, as indicated in Table A. The latest gradings shown in the table were adopted in 1946 and are serving reasonably well to make for the best utilization of locally available material. Recent studies, which are briefly discussed in the following paragraphs, indicate that on selected projects it may be desirable to change the upper plasticity index limit.

Pumping survey

One of the primary functions of subbases under concrete pavements is the prevention of pumping. The Ohio highway department is now completing the analysis of data covering a survey of pumping, made in cooperation with the Portland Cement Assn., on approximately 500 miles of portland cement concrete pavement. Some of the tentative conclusions are:

1. Little pumping occurred on the majority of projects carrying 50 and less 14,000-lb axles, and 20 and less 18,000-lb axles, per 8 hours even under unfavorable conditions of subgrade soil and design. It appears that careful consideration should be given to the possible omission of granular subbases for the prevention of pumping where expected volume of axle loadings is within these limits.

2. Where the number of 14,000-lb axles per 8 hours is expected to be within 51 and 250, it may be well to consider the use of a granular subbase even though it is not a first-class,

TABLE A—GRANULAR SUBBASE GRADINGS USED IN OHIO, 1938-1950

Sieve	Total percent passing											
	1938	1942 Gradings			1944 Gradings				Present gradings*			
		1	2	3	A	B	C	D	A	B	C	D
3-in.		100	100		100				100	100	100	
2-in.	100			100	30–70	100	100	100	30–70	90–100	90–100	100
1-in.		30–70	75–100			70–100	70–100			70–100	70–100	
½-in.			50–90			40–100				40–100		
¾-in.					15–40				15–40			
No. 4						0–40				0–40		
No. 10	50	0–25	25–70	50–100	0–15	0–15	35–75	50–90	0–15	0–25	25–75	40–100
No. 50							0–10	0–20			0–20	0–25
No. 200	0–25		0–15	0–15								
Liquid limit	0–35	0–35			0–35				0–35			
Plasticity index	0–15	0–10			0–10				0–10			

*Adopted in 1946.

low-plasticity material. Traffic data indicates that this load group would include 20 to 80 axles of 18,000 lb and greater.

3. The study shows that granular subbase material having a plasticity index of 6 or less should be used over fine grained subgrade soils to prevent pumping where the traffic is expected to have over 250 axles of 14,000 lb per 8 hours. Traffic counts indicate that this volume of trucks would include more than 80 axles of 18,000 lb or greater.

In this study the maximum axle load was 32,500 lb; the average axle weight of the 18,000-lb and over group was 20,200 lb.

The cost of maintaining relatively few pumping joints over the life of the pavement must be carefully weighed against the cost of a granular subbase or other forms of subgrade treatment.

In the writers' opinion, one of the most important problems facing highway engineers today is the determination of the effect of various axle loads and gross weights on different pavement designs on various subbases and subsoils. This problem is equally important for both rigid and flexible pavement.

In December 1949, representatives of 13 states, the Bureau of Public Roads and the Council of State Governments organized the Interregional Council on Highway Transportation. The council instigated tests on a concrete pavement in Maryland subjected to various controlled single and tandem axle loads. The data gathered on this project should be of considerable interest and value to all highway departments. However, it can only be considered as a first step since the project represents only one of the many pavement designs used in the various states.

By L. A. PALMER*

Airport and highway subgrade problems

In its broad sense, pavement includes highways and airport runways, taxiways, parking areas, warm-up aprons and seaplane ramps. Normally, highway pavements are subjected to transient loads. On the other hand static loads may continue for hours or even days and weeks on airport parking areas. The consolidation of saturated fine-grained soils as caused by the egress of water under applied load is a slow process. Transient loads do not tend to effect this type of consolidation. In highways such consolidation tends to be caused only by the dead weight of fills on the marshy ground. In the parking areas at air stations this type of consolidation often results from wheel loads and the resulting uneven settlement of the pavement often necessitates costly repair and maintenance.

Again, subsurface drainage, which often is entirely feasible for highways, is generally much less effective in more extensive paved areas such as wide runways. The subgrade moisture content is generally highest in areas nearest the edges of runways. It is here that perched water tables, due to poor shoulder grading, are most frequent. But perched water tables tend to occur under the entire paved widths of highways. The upward percolation of water under hydrostatic head and under highway pavements in

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cut sections of valley walls of streams is especially conducive to pumping and frost heave. This condition is peculiar to highways.

The progressive densification of granular base courses and subgrades under traffic is common to both highway and airport pavements but the problem is more acute at airports where the combined effects of static load and vibration tends to accelerate the compaction of relatively cohensionless granular material.

On the whole, subgrade problems have been increased, both in number and degree, with the advent of extensive airport paving. Concrete is one of the most standard of all paving materials and great progress has been made in the design and construction of concrete pavement itself. In theory alone, design of the slab is a simple matter, assuming that the materials supporting it could be unchanging over the years with respect to their many physical properties. If it were not for one thing, water, this would be almost true. The really difficult job in concrete pavement design is to anticipate and overcome the many and diverse soil problems. Ultimately, all design criteria for pavements will be written essentially around these problems.

Considering the entire 48 states, about half of a great continent, the range in conditions, both with respect to materials and climate, is too great to permit extensive generalizations in design criteria that can be equally applicable to adequate and economical design of concrete pavements throughout the nation. The first step is simplification, division of the United States into separate, major physiographic areas and to then gradually and systematically develop the design criteria for each separate major division. This plan warrants the serious consideration of all who are confronted with the problems of pavement design. Certainly it can never be good engineering to recommend base courses of granular material in one area simply for the reason that such base courses have been used effectively and economically in other areas. The local availability of such materials must first be considered.

Subgrade moduli and slab thickness design

Wherever idealized and constant subgrade conditions may prevail, the design of slab thickness, based on theory involving the assumption that the slab deflection caused by load is proportional to a modulus, K, characteristic of the supporting earth, is a thoroughly sound procedure. But there are two essential requirements if and when such theory may be applicable; the subgrade support must be permanent and it must be uniform. The theory cannot apply to a slab supported on subgrade "mushrooms." It is not applicable to slab corners and edges that have no support because of progressive densification of granular material or progressive escape of fine-grained soil particles by pumping. If there were no joints in concrete pavement the problem would be simpler. Because of joints, the subgrade reaction varies with respect to the position of the wheel load on the slab. Because of this variation in pressure intensity on the subgrade, the latter is compacted to a variable

and not to a uniform extent and the slab becomes nonuniformly or only partially supported.

Where settlement by consolidation of underlying, fine-grained soil strata is caused either by the weight of overlying fill or by earth pressures resulting from the superposing of multiple wheel loads as in a parking area, the subsidence is almost never uniform. The two theories that have been advanced, the older one of Westergaard[52] in which the subgrade is regarded as a dense liquid and the more recent theory as advanced independently by D. L. Holl* and A. H. Hogg† in which the subgrade is considered to be a semi-infinite elastic solid, have reference to only that type of yielding of the subgrade which results instantaneously from a load applied to the slab. They have no reference to the slow and progressive subsidence brought about by densification of granular layers and consolidation of fine-grained soils through egress of pore water. The combination of base course and subgrade acts neither as a dense liquid nor as an elastic solid except under comparatively rare conditions. For the most part, the progressive deformation or subsidence tends to be nonrecoverable. Again, neither Westergaard's K nor the subgrade elastic modulus, E, of Holl and Hogg can be expected to be constant with time in view of the changing subgrade moisture conditions.

Finally, there are serious difficulties experienced in the evaluation of these moduli. The effect of plate size in loading tests, designed to yield a K or E value, must be considered from the standpoint of lack of homogeneity, with depth, of the supporting earth materials. For example, a hard layer existing 3 or 4 ft below the base course will have relatively more effect when loading with a large plate. On the other hand, if the top foot of subgrade is given extra compaction and is considerably firmer than the material under it, the reverse is true. As a rule, we are dealing with multiple layered and nonhomogeneous materials that are not amenable to precise analysis.

Despite all these difficulties and limitations, the theories have been and will continue to be extremely useful. For example, it is clear from the Westergaard theory that whether the K is 200 or 400 lb per cu in. is of relatively less importance than the thickness or strength of the concrete, provided there is uniform and permanent slab support in both cases. It may be more economical therefore to compensate for a weak subgrade by thickening the slab than to construct a base course, especially where there is a dearth of available base course materials.

The problem of "soil control," bringing about those desirable subgrade conditions that can assure both permanency and uniformity of support, remains the most serious and difficult of all of the problems involved in the design of concrete pavements.

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*Holl, D. L., "Thin Plates on Elastic Foundations," *Proceedings*, Fifth International Congress for Applied Mechanics, 1939, pp. 71-74.

†Hogg, A. H. A., "Equilibrium of a Thin Plate, Symmetrically Loaded, Resting on an Elastic Foundation of Infinite Depth," *Phil. Mag.*, Mar. 1938, No. 7, V. 25, pp. 576-582.

By TILTON E. SHELBURNE and PHILLIP L. MELVILLE*

It is indeed gratifying to find a paper like Woods' which presents such a wealth of practical and factual information. The ability to look at the pavement far enough ahead, to understand its basic functional components and to do so above political boundaries is a rare gift to be prized.

The Virginia Department of Highways has been interested for some time in collecting data relating to the influence of subgrade soil upon pavement performance. Results of these studies give pertinent information which applies to Virginia conditions.

In 1947, R. L. Sheppe reported† the results of a reconnaissance of 440 miles of concrete pavements throughout the state. The purpose of this study was to summarize data on design, construction practices, sources of materials and to determine by field surveys the relative performance of the individual projects. While the survey brought to light many items concerning the effects of traffic, climate, pavement design, construction practices and the use of materials, results relating to the influence of subgrade soils on pavement performance are particularly pertinent to this discussion. An excellent correlation was found between the severity of pumping and the general soil area in which the pavement was located. For example, in the Coastal Plain (mostly granular and well-drained soils) where 75 percent of the rigid pavement mileage in the state is located, less than 9 percent of it showed signs of pumping. In the Piedmont Plateau (more plastic and poorly drained soils derived mostly from igneous and metamorphic formations) where 22 percent of the mileage is found, 43.7 percent of it was pumping. Finally, in the Appalachian Valley (plastic silty clay soils derived from sedimentary formations) where only 3 percent of the rigid pavements are located, 71 percent of the mileage was found to be pumping.

A study of the roads in southern Albermarle County permitted a comparison of the influence of soils upon pavement performance and maintenance costs.‡ In the analysis it was found that over a period of years more maintenance and a higher type pavement were required for primary highways located in the Triassic basin than on adjacent soils derived from Pre-Cambrian formations. Over a period of years the average maintenance cost of secondary roads was also higher when built in the former area as compared to similar routes in the latter area.

A third study consisted of a state-wide performance survey§ made on 18,000 miles of pavements in Virginia at the time of the severe 1948 spring break-up. The survey was not confined to rigid pavements, but included 8500 miles of primary (mostly flexible pavements) and 9500 miles of hard-surface secondary roads. For purposes of analysis the state was divided

*Director of Research and Associate Research Engineer, respectively, Council of Highway Investigation and Research, Charlottesville, Va.

†Sheppe, R. L., "Progress Report No. 1—A Reconnaissance of Portland Cement Concrete Pavements in Virginia," *Unpublished report*, Virginia Dept. of Highways, June 1947.

‡Melville, P. L., "An Airphoto Study of the Triassic Area in Albemarle County," *Bulletin* 13, Highway Research Board, 1948.

§Stevens, J. C., Maner, A. W., and Shelburne, T. E., "Pavement Performance Correlated with Soil Areas," *Proceedings*, Highway Research Board, 1949.

into 12 general soil areas, based upon physiography, geology and pedology. The data permitted a study of the relationship between pavement performance and the general soil area. Despite the fact that in certain areas roads had been built to higher standards than in others to provide against undesirable soil characteristics, the analysis revealed that pavement performance was directly related to the soil area in which it was located. Again, the Coastal Plain was found to be one of the better areas on the basis of pavement performance. Considering only the primary roads in this area, 67 percent of the mileage was rated good, 23.3 percent fair and only 9.7 percent poor. In contrast, performance of roads in the Triassic "red bed" soil areas was much poorer. For example, ratings for primary roads in this area were: 28 percent good, 16 percent fair and 56 percent poor. These typical data are used to illustrate the importance of the soil area as a major factor influencing pavement performance.

The three studies briefly described above substantiate and confirm Woods' conclusion that "on the basis of the distribution of soils and materials of construction, it seems logical to approach the problem of design with the regional-soil concept in mind."

If we leave the national for the international level, we find that the deep concern in the problem of rigid pavement design is not exclusively "made in U. S. A." During a visit to Europe in 1949, the junior author of this discussion found a similar transatlantic situation.

In France in particular he found much thought devoted along entirely new lines. For example, engineers are likely to hear more about thin pavement slabs made of prestressed concrete. Also a promising method is known as the "caisson pavement," composed of a thin concrete course as subbase, a "sandwich" of selected compacted local materials of variable depth and a surface on top. The lower layer is supposed primarily to seal off moisture, the middle layer to distribute the load and the top one is only a wearing course. Under extremely unfavorable conditions due to traffic, climate or materials, small vertical retaining walls have been placed on the outside of the sandwich.

No doubt other solutions exist or will be found. It is a comforting thought to find so much fruitful research being carried on for better highways around the world.

By WILLIAM VAN BREEMEN*

The author, in his valuable and thought-provoking paper has appropriately brought to the attention of the highway profession a number of items relating to the problems of pavement design that warrant serious consideration.

As pointed out by Woods, the serious structural failure of many rigid pavements in recent years as a result of large concentrations of heavy truck traffic indicate the need for continued intensive research in the field of subgrade and subbase materials.

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As matters stand, there is an abundance of evidence to show that performance of any pavement depends to a great extent upon the characteristics of the subgrade material upon which it has been constructed. This is particularly the case in connection with pavements that carry heavy truck traffic. In New Jersey, as in many other states, this fact is evident from the variable amounts of damage found on the heavy trucking routes, even under conditions where the factors of pavement design, concrete strength, length of service and volume of traffic are essentially the same. Without exception, the most serious failures are found in connection with pavements constructed directly on fine-grained, relatively impervious soils, such as the clays and silts. These soils are the typical "pumpers" that have contributed materially to the extremely serious pumping and faulting of joints throughout a large part of the United States during the past ten years, and in New Jersey since 1931. With an increase in the granular content of the soil—that is, an increase in the amount of sand, gravel or stone—the amount of damage is found to be proportionately less. In connection with subgrades that are almost totally granular there is little or no pumping and relatively little damage.

Building for heavy traffic

Twelve years ago, convinced of the important effect of the subgrade material on pavement performance, New Jersey abandoned the construction of concrete pavements directly on clayey, silty soils and adopted the practice of covering all soils of this type with a layer of relatively clean, granular subbase material. Subbases having a minimum thickness of 12 in. are now employed on all heavy trucking routes. These subbases also extend under the shoulders, which are generally not less than 10 ft wide. Inasmuch as clean, granular materials, especially sands, are difficult to compact, the present specifications require that the upper 5 in. of the subbase shall have a gravel content of at least 45 percent and an elutriable content of from 4½ to 7 percent. This method of construction appears to have satisfactorily solved the compaction problem.

The employment of granular subbase has had two important beneficial effects. First, it appears to have prevented pumping. Second, it has effected a marked reduction in cracking. Precise levels taken on the pavement at close intervals indicate that the reduction in cracking is due primarily to a reduction in the magnitude of differential heaving during freezing weather. The greater stability of the granular subbases during periods of thaw may also be a contributing factor.

It is necessary to point out, however, that in New Jersey the solution to the problem of designing pavements that will resist the damaging effects of heavy truck traffic has not been sought through improvements in subgrades alone. On the contrary, as a means of strengthening the pavement itself, it is the current practice to install sturdy load-transfer devices at all joints—there being ample evidence that under heavy truck traffic, and de-

spite all reasonable improvements in subgrade, it is absolutely necessary to adequately dowel all joints, otherwise an excessive amount of faulting will sooner or later occur. Furthermore, reinforcing steel is installed to prevent the opening of cracks, and thus prevent the faulting that would be certain to occur if they were free to open.

Loss in stability during thaws

As indicated by Woods, the major problems pertaining to the prevention of frost-heaving have perhaps been solved. It appears, however, that there is a great need for further intensive research in connection with loss in stability during periods of thaw. The familiar spring "break-up" most often found in connection with so-called "flexible pavement" is an example of this loss in stability. Studies made in New Jersey indicate, furthermore, that many of the granular materials are not immune to this action. This was proved experimentally during the winter of 1947-1948 by means of weighted steel plungers, the lower ends of which were in contact with the various materials being tested. Measurements made at frequent intervals indicated the amount of penetration was much more pronounced during thawing than at any other time. There were in fact a number of instances where no penetration occured except during thaws. These tests were, of course, merely indicative. But it is evident that in the colder parts of the country the modulus of subgrade reaction is a factor that is subject to considerable change throughout the year. And it is furthermore evident that any dependable theoretical design procedure would have to take this phenomenon into account.

Woods recommends that caution be exercised in the selection, use and construction of granular subbases, particularly in regard to sand. In New Jersey, his cautionary remarks are substantiated by the performance of several sections of pavement constructed on thick layers of sandy material. Especially on fills, a pronounced settlement at the joints has developed due, apparently, to further consolidation, rearrangement or lateral displacement. Lack of proper consolidation during construction may also have been a contributing factor. In any case, the writer is of the opinion that, with respect to sands, the greater the depth the greater the need for caution.

Difficulties in pavement design

In his discussion on subgrade modulus, Woods points out a number of difficulties standing in the way of a theoretical approach to pavement design—and rightly so. Among them, the phenomenon of pumping has brought us to a realization that subgrades cannot be evaluated solely in terms of the results of loading tests. On the contrary, we now know that pumping is basically an erosive action wherein the soil particles at the surface of the subgrade are carried away by free water that has been agitated and subsequently ejected by the passage of traffic. Actually, many fine-grained soils have the capacity to withstand the loads applied to them, but because they lack resistance to erosion they fall in the category of "pumpers."

It consequently is evident that in the evaluation of subgrade soils "resistance to erosion" is a highly important factor.

Warranting serious thought on the part of every highway engineer is that portion of Woods' paper in which he points out the major differences in soils, climate, materials and traffic that exist throughout the United States. In line with the "regional soil concept" which he proposes as an approach to pavement design, it is clear that there should be an abandonment of all attempts to effect uniformity in design practice on a nation-wide basis and that, because of these major differences, the most that should rightfully be hoped for is uniformity on a regional basis.

For years, the engineers throughout the various parts of the United States have had pronounced differences of opinion in regard to the proper design of concrete pavements; such details as the proper spacing of joints and the inclusion or omission of expansion joints have been controversial matters that have never been settled to the satisfaction of anyone. If, on the other hand, engineers become aware of the full significance of the regional concept propounded by Woods they will appreciate that their own personal judgment is necessarily based, in the main, on the conditions that exist within their own particular sphere of activity. They will furthermore appreciate that what may be the best practice in one part of the country may not by any means be the best practice in another part. At the moment, these controversial items often tend more to keep engineers apart, than to bring them together. It is hoped that with the adoption of the regional concept these difficulties will have become resolved.

Solutions to the special problems facing the inspector of mass concrete work are emphasized in

Inspection of Mass and Related Concrete Construction*

By LEWIS H. TUTHILL†

SYNOPSIS

This paper points out that inspection of any kind can be no more effective than that permitted by the specifications and, particularly, by the established job standard of inspection. No distinction is made between the usual activities of inspection and those associated with concrete control since both have the same objective: a serviceable and presentable structure. Details of requirements and procedure to this end are discussed.

INTRODUCTION

Discussions on inspection too often appear to be based on the premise that if we could get inspectors, who were just what we wanted them to be personally, to do just what we wanted them to do on the job, results would be close to perfection. This would help, of course, but there are several other factors which have an immense bearing on the effectiveness of both the individual inspector and his inspection organization. These factors apply to any concrete inspection work and it is therefore appropriate to discuss them in connection with inspection of mass concrete.

FACTORS INFLUENCING EFFECTIVENESS OF INSPECTOR

Specifications

Inadequate and indefinite specifications place the inspector at a disadvantage. Although a good job is usually the intent of specifications there can be a variety of interpretation of "a good job" and what it takes to get it. So far as possible the specifications should state specific requirements for materials, procedure, equipment, finishes, and tolerances and it is assumed that the specification writer will know what

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these specific requirements should be. Wherever choice of equipment or method can be left to the discretion of the contractor this choice should be left open. However, experience has shown that there are many details of equipment and procedure which must be specified if best results are to be assured. Typical of these are: vibration, weight batching, finish screening at batch plant, sandblast cleanup, controllable buckets for placing, power floats, etc. For instance, the aggregate specifications should include the sentence, "Finish screening of the coarse aggregate shall be performed at the batching plant by means of horizontal vibrating screens and the aggregate shall pass directly from the finish screens to the batching plant bins without intermediate storage. Aggregates larger than 1½-in. shall be lowered into the bins by means of rock ladders which shall operate effectively to keep breakage at a minimum." When only the character of results are specified, it is often too late to change either procedure or equipment after less than desired results are obtained. In such cases the inspector's hands are tied unless he can persuade the contractor to use preferred time-tried procedures from the start. This is expecting too much of inspection.

Management policy

A forthright attitude by management toward inspection is sometimes lacking. This is revealed nowhere more definitely than in the degree of support and backing afforded the engineering and inspection organization. When certainty of support is lacking, inspection is correspondingly ineffective regardless of its potential quality. Failure to support inspection or an attitude of leniency on the part of management may stem from the mistaken belief that proper inspection increases costs; although actually the reverse is true, especially in the long run. Also, sometimes, through failure to live up to its own part of the contract in some respect, or through necessity for negotiating changes or earlier completion, management is at a disadvantage when it comes to requiring the contractor to perform strictly up to requirements.

Ability of contractors

Difference in ability of contractors has much to do with the quality of workmanship obtained regardless of the quality of inspection. Poor performance may be due either to the contractor's inability to secure certain types of material or equipment, or to certain attitudes and lack of skill among his employees. During the war, as well as afterward, there have been many situations where such job conditions have largely nullified the best efforts of capable inspection forces.

Immediate supervision

The immediate supervision of the inspector has much to do with his success and effectiveness. This is evident in the obvious improvement

in his work when the following responsibilities of the supervisor to the inspector are recognized and acted upon.

1. To outline in advance those matters for which each inspector in his charge will be directly responsible.

2. To inform his inspectors as to decisions which are to be referred to the supervisor.

3. To see that his inspectors have not only all necessary instructions pertaining to their work but also information concerning special problems and conditions which are likely to arise.

4. To see that an inspector is not placed on the job without knowledge of special arrangements or agreements that have been made with the contractor and that affect the work with which the inspector is concerned. Particularly, the inspector should be informed at once of any concessions or special interpretations of the specifications.

5. To see that his inspectors are made aware that they have their supervisor's full support in all proper execution of their inspection work.

6. Whenever practicable (and it is seldom that it is not), to obtain the opinions of inspectors and engineers close to the job before reaching decisions relative to requests and representations made by the contractor.

Also, occasional joint meetings of the inspectors and their supervisors, for open discussion of job problems, practices, and requirements will be found beneficial.

Concrete control

Lack of recognition of the benefits of effective concrete control activities, especially on larger projects, sometimes deprives the inspection organization of a valuable facility. In its narrowest sense, concrete control is considered by some to be concerned only with concrete materials, batching and mixing, and related testing. Actually, effective concrete production control is the basis of good concrete inspection, and is an important integral element of it. No distinction should be made between inspection and concrete control activities or between the engineers engaged in them. Both activities have the same objectives in a presentable and serviceable concrete job. All the activities of concrete control and inspection should be in one organizational group under the direction of an experienced concrete engineer and construction man who is fully aware of the technical, as well as the practical, aspects of concrete production and construction. As long as he has these qualifications and the responsibility for all phases of the concrete work, his title is immaterial. Inspection at Hungry Horse Dam is organized on this basis.

PROCEDURE AND FACILITY REQUIREMENTS FOR EFFECTIVE INSPECTION OF MASS CONCRETE

Usually the volume of mass concrete, when used, is sufficient to make many refinements of procedure and equipment even more practical

than they often are considered to be for less extensive work. Specifying such procedures and facilities will go a long way toward insuring the effectiveness of inspection on mass concrete work. Some of the most important of these are:

1. More detailed exploration and testing of the aggregate sources.
2. Separation of the aggregate into at least five sizes including sand.
3. Reclassification and processing of the sand to more exacting grading limits.
4. Unfailingly thorough drainage of sand.
5. Finish screening of coarse aggregate at the batch plant with the finished material passing directly to the bins without intermediate transfer, handling, or stockpiling.
6. Rock ladders.
7. Self-cleaning batcher bins filled by a vertical drop over the opening.
8. Automatic weight batching.
9. Automatic recording of batching and mixing operations.
10. Prevention of initial concrete temperatures above 70 F at most.
11. Effective mechanical equipment for sampling aggregate and concrete.
12. Mixer performance tests.
13. Rigid requirements for cleanup and mortar coating of horizontal construction joints.
14. Correctly performing buckets for transporting and placing the concrete.
15. Thorough consolidation by effective internal vibration.
16. Positive provisions for thorough water curing.

Aggregate preparation

Preparation of the aggregates should be included in the construction contract. Unless it is, it may be impossible to obtain the quality of concrete desired. In this way, the same contractor is responsible for both production and use of the aggregates, and will integrate the two operations much more effectively than could possibly be done with two independently responsible contractors. If he is to use the aggregate, the contractor will be interested in details of preparation and handling that contribute to the uniformity and workability of the concrete; whereas if he is merely producing it, he will be interested solely in production.

Because mass concrete contains less cement and sand, the grading of sand is more important than otherwise as a factor in securing good workability. Primarily, this means an ample percentage of the several sizes of fines from 200- to 50-mesh, about 20 percent altogether passing the 50-mesh screen. Best results will be obtained when other sizes are fairly well proportioned, particularly without excess in any of the larger sizes. Such a sand might well be graded: No. 8, 15 percent; No. 16, 18 percent; No. 30, 22 percent; No. 50, 25 percent; No. 100, 15 percent; and pan, 5 percent. Hydraulic separators are employed efficiently for processing the quantities of sand required for large mass concrete jobs.

This equipment will roughly separate the sand into a number of fractions, which can be recombined in the proportions necessary to create the required grading. At most, three fractions will usually suffice, as this grading will permit removal of part of an intermediate fraction which may be in excess. With the use of intentionally entrained air in the concrete, it is not necessary to go as far with these refinements in sand grading.

In preparation of the coarse aggregate for mass concrete dams, the special feature is the screening and handling of large aggregate up to 6 in. and sometimes 8 in. in diameter (Fig. 1). Because there are fewer openings per square foot in the screens on which the large sizes are retained, and because these sizes, being heavier, break up more in handling, significant undersize tends to overrun allowable limits more readily in these than in smaller sizes. These tendencies are best controlled by avoiding all possible drops and transfers after the material, whether it be crushed rock or gravel, has received finish screening on screens of ample area. For these reasons, most advanced specifications now require that the finish screening of coarse aggregate be performed at or near the batching plant and that the finished material pass directly to the batcher bins without intermediate transfer, handling, or stockpiling. Effective rock ladders (Fig. 2) should be provided to lower the material into the bins in easy stages with a minimum of impact and resultant breakage.

Mixing

For this scale of concrete production in mixers of 2- and 4-yard capacity, it is worth while to make tests of mixer performance as discussed in the *Concrete Manual**, Designation 26, to insure that maximum uniformity is obtained in the mixing period, that the sequence of batching and introduction of the ingredients of the batch are most favorable to best mixing efficiency, and that the mixing action is main-

*Publication of the U. S. Bureau of Reclamation, Fourth Edition.

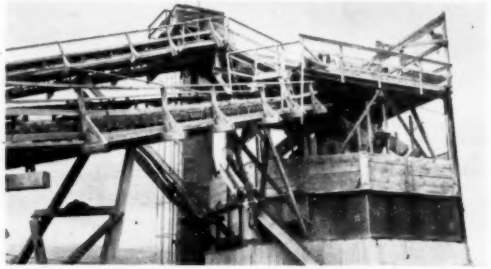

Fig. 1—Unusual uniformity of concrete was obtained at Angostura Dam as a result of finish screening the coarse aggregate on this horizontal vibrating screen over the batching plant bins.

Fig. 2—Rock ladders prevent excessive breakage and resultant undersize by lowering large aggregates from finish screens to batching plant bins at Angostura Dam.

tained up to required standards by proper attention to blade wear and cleaning of the mixers.

In planning and erecting the mixing plant, every effort should be made to arrange the batching and mixing controls in such a position that, by means of proper gratings and lighting, the plant operator and inspector can see into, and discern the consistency of concrete in at least one of the mixers being operated. Both operators and inspectors have found this arrangement a great help in producing concrete of uniform consistency.

To facilitate sampling for control tests in large batching and mixing plants, it has been found necessary to require effective and safe devices for taking samples of each size of aggregate as batched, and of the concrete as mixed (Fig. 3). With the use of entrained air, it is necessary to include tests of concrete sampled after placing and vibration, in order to properly measure the properties of the concrete as placed.

Uniform mix

The small amount of mortar in mass concrete is the source of many of its special advantages, such as lower cost and lower generation of heat and temperature rise due to the lower than usual cement content. The correspondingly lower water content reduces shrinkage on drying and improves durability. The water requirement is further reduced by a low initial temperature of the concrete. However, to obtain the full measure of these benefits, it is necessary to keep sand content minimum. The percentage of sand required is surprisingly low, especially with en-

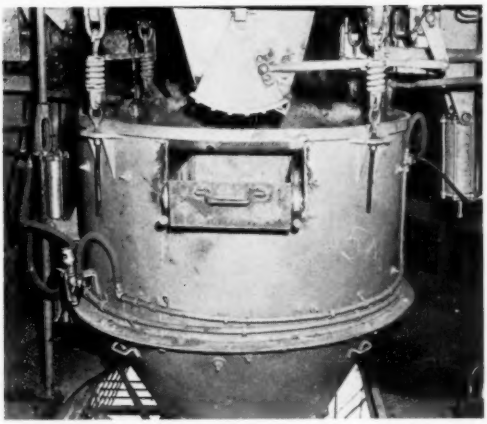

Fig. 3—Good inspection is much easier with good facilities for sampling aggreates as batched.

trained air. Tests should be made to verify that the practical minimum percent of sand is being used, by observing at what percent lesser amounts produce dangerously unworkable mixes. Surplus workability should be avoided; it means a concrete of lower quality, less durability, and more shrinkage on drying.

The major factor in keeping the sand content at low minimum values is the uniformity with which the concrete can be produced, handled and placed. Elements contributing to this are:

1. Finish screening at the batch plant.
2. Good drainage of sand.
3. Rock ladders.
4. Self-cleaning bins frequently emptied.
5. Accurate batching.
6. Good mixing.
7. Handling without appreciable separation or slump loss.
8. Strong, effective vibration.

With these factors effectively contributing to a high level of uniformity, and with good ample vibration to consolidate the concrete in place, only narrow margins of extra sand and mortar are required to offset variations in workability.

A common problem that requires careful attention in placing concrete in large dams where freezing will occur is the placement of concrete somewhat richer in cement in the upstream and downstream faces of the dam. Six feet is regarded as ample to provide the required added protection against weathering, but the thickness obtained is usually a matter of construction expediency depending on the placing and dispatching facilities, the size of buckets used, and whether they can dump

a part of a batch. On jobs where a lean interior mix and a richer exterior mix are used the inspectors must be on guard against superfluous use of the richer mix. In many cases where there is an inclination to use it, such as during threatening or showery weather, or around penstocks or other features embedded in the interior, the benefits will not justify the added cost of cement and heat removal.

Handling

Since the practicable and preferable consistency for mass concrete is between 1½ and 2 in. of slump, chuting systems for placing the concrete are not practicable, so large concrete buckets holding 2, 4, or 8 cu yd are used. Common failings of some buckets used for this work are: (1) too small an opening to discharge readily concrete of the low slump otherwise satisfactory and conveniently placed with vibration; and (2) inability to discharge only a part of the batch in two or more places, or to discharge the batch more slowly, in confined spaces, along the forms, in corners, in reinforcing, etc. The size of the bucket is unimportant, so long as it is filled in a manner that will not separate the large rock from the concrete; but it should discharge low-slump concrete readily and be capable of dividing the bucketful among several locations.

Placing

Forms—Forms for mass concrete differ from ordinary forms in that they depend for their security on anchorage in the previous lift of concrete (Fig. 4). Cantilever forms depend on an anchor near the top of the formed face of the previous lift of concrete, while other forms are braced and tied to anchors (hairpins) in the horizontal top surface of the previous lift. Consequently, with a new lift of concrete usually being added when the lift below is not more than 3 days old, form an-

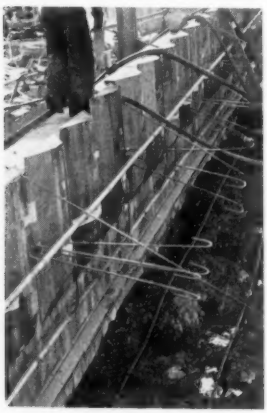

Fig. 4—Form anchor requirements vary with design and height of form, interval between lifts, early strength of concrete and temperature.

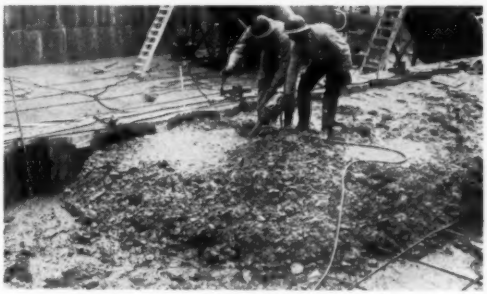

Fig. 5—Good vibration permits use of efficient mass concrete with low cement and water content.

chors must be ample in size and bearing area, and be deeply embedded, because of the relatively low strength of mass concrete at this early age, especially in cold weather. Cement content cannot be increased over otherwise optimum amounts or a less favorable type of cement used, merely to save on form anchorage.

Vibration—Vibration of mass concrete should leave no doubt as to its being fully consolidated, batch by batch, as the concrete is placed (Fig. 5). Usually, it is customary to revibrate the concrete against the forms around the perimeter of each block; and as a result, few rock-pockets are found when the forms are removed. Consequently, the absence of rock-pockets on the face is not proof that full consolidation is obtained within the block. This can be determined only by watchful inspection as the concrete is placed and vibrated, and by establishing procedures for this vibration that leave no portions of doubtful consolidation.

The portions sometimes in doubt are the outward edges of the batches as deposited which have to wait for some time before concrete will be placed against them. Sometimes these outward edges are left unvibrated until the next advance in the placing operations causes a bucketful of concrete to be placed against them. Often by that time the unvibrated concrete at the edges becomes too hard to consolidate properly, or there is a lack of systematic vibration that will unmistakably consolidate the older concrete and newer batch when the older concrete can still be vibrated. Consequently, it is best to vibrate fully all parts of each bucketful when it is dumped, sloping the forward edge about 4:1 or flatter as necessary, to avoid flow on the slopes and without overrunning the lower slopes. Thus, no matter what may happen, all concrete in place is fully consolidated. Overvibration of concrete of this low slump is unlikely. Evidence of overvibration should be regarded as evidence that the amount of mixing water should be reduced, not the amount of vibration.

Fig. 6—The construction joint surface is more easily cleaned with the sandblast when rocks are embedded and footprints are eliminated by a workman on "snowshoes."

Cold joints in such concrete in warm weather or following a delay can be eliminated by full systematic vibration of the new concrete close to the entire area of contact with the older concrete. This procedure is much cheaper and better than using a mortar batch every time this happens. The line of a cold joint, so treated by vibration, may be found on the formed surface, but it cannot be found by core-drills. Although too hard to revibrate, the older concrete is generally very green and, if it is prevented from surface drying by an occasional fine light spray or by a temporary cover of wet burlap, there is no question of the completeness of bond with new concrete thoroughly vibrated at the contact surface.

Except for increased size and power, vibrators and their inspection are no different for mass concrete than for other concrete. Vibrators capable of 6000 vibrations per minute are necessary for efficient results. Electrically-driven vibrators have the advantage of usually working up to performance standards or quitting altogether, whereas air-driven machines will slow down when not in good condition or the air supply is short. Vibrators should receive systematic servicing after each shift to insure the most uninterrupted efficient performance.

Before the top surface of each lift sets, it should be gone over by a man wearing wooden "snowshoes" which give him two or three times the area of support provided by ordinary boots or shoes (Fig. 6). Using a small immersion-type vibrator as he steps on protruding or unembedded larger pieces or rock, he can embed them with their top surfaces at the level of the surface of the lift and eliminate footprint depressions. This procedure leaves an even surface that can be cleaned much better and more efficiently with the wet sandblast, and is well worth doing.

Cleanup and mortar coating—The sandblast cleanup and mortar coating of horizontal construction joints in mass concrete is no different in principle than for such joints in other concrete; but in mass concrete, the large areas and splendid accessibility of these joints afford an opportunity to exercise fully the best practices and thus obtain the finest results in watertight, well-bonded joints. A deep cut with the sandblast is not necessary to get a good joint surface on concrete of this

Fig. 7—Wet sandblasting and washing, as a final operation, is the most efficient way to get a clean joint surface.

low slump. Only the surface film need be removed. Coarse aggregate does not necessarily have to be exposed. In the sandblast cleanup operations (Fig. 7) it is mainly a matter of careful inspection to see that no areas are missed, particularly along the edges of the blocks, and to see that, in the washing following the sandblast, all sand and debris are removed from edges, corners, and low spots.

The mortar coat on the joint should not be thick and should not be allowed to crowd heavily into outside corners. The clean surface of the joint on which the mortar is placed should have been wet and have approached a damp condition, free of puddles and visible surface moisture. Vigorous and thorough scrubbing in of the mortar as it is spread has much to do with securing good bond and should not be overlooked.

Curing and protection of mass concrete presents no special problems not common to these operations on other work.

CONCLUSION

The technical details of concrete inspection can be found in the many published manuals on this subject and in various papers and committee reports on concrete control and construction practice. However, discussions of factors affecting the effectiveness of inspectors are not as readily found. On a mass concrete job, or in fact on any concrete job of sufficient importance to merit inspection, the inspection force will usually be found to be competent, at least potentially. Any apparent lack of effectiveness will usually be the result of job conditions beyond its control. Where effective inspection is wanted, management must see that the specifications and the setup of the job are such that inspection can be effective.

Portland cement grout proved effective in controlling seepage into excavation areas for Chief Joseph Dam.

Experimental Grouting Investigation for Chief Joseph Dam*

By J. M. WELLS†

SYNOPSIS

This paper describes the development of a method for the control of seepage into the excavation areas of the right abutment, and through the right abutment, at the Chief Joseph Dam project on the upper Columbia River in the State of Washington.

Laboratory studies are presented on various types of grout mixtures, as well as techniques and procedures for drilling in coarse gravel by jetting methods, procedure for grouting pervious gravels and the efficacy of grouting treatment.

INTRODUCTION

The overburden in the right bank of the Columbia River consists of a layer of pervious gravel 110 ft thick resting on bedrock and overlain by 180 ft of glacial till. Field and laboratory tests of the gravel formation indicated that some measures must be taken to limit seepage into the right abutment excavation during the first stage of construction of Chief Joseph Dam to quantities which could be economically pumped. Controlling seepage through grouting of the gravels appeared feasible. Investigation of the control of seepage by grouting involved (1) development of a grouting mixture capable of penetrating the materials of the pervious formation, (2) an economical method of drilling a large number of holes in the gravel, (3) a procedure of introducing the grout mixture into the pervious material and (4) the determination of the efficacy of the grouting.

Exploration of the gravel stratum by churn drill holes, test pits and tunnels disclosed the predominant formation under the till to be an open work gravel with varying amounts of clay in the voids, interspersed with cracks up to ⅛ in. in width. Seepage through the cracks in this forma-

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tion, as evidenced by pumping tests in drill holes and test pits, was extremely high. Other materials found in lesser degree were large boulder layers with sand and clay in the voids, well graded gravel and coarse sand. Exploration to date has disclosed no predictable bedding of these materials. The coarse sandy lenses were relatively thin and frequently inclined.

Observations of the materials in the pervious stratum suggested that a more penetrating mixture than portland cement grout would be required for seepage control and that drilling of holes by jetting methods might be possible.

LABORATORY STUDIES OF GROUT MIXTURES

The laboratory investigation was conducted to develop a grouting mixture which would (1) have high penetrating qualities in well graded sand and gravel and in openwork gravels with ordinary low pressure grouting methods, (2) have fairly high resistance to water erosion, (3) have setting time controllable by suitable admixtures and (4) develop sufficient strength to provide stability of slopes in the gravels below river level.

The materials chosen for testing and study were: (1) portland cement, (2) Lumnite cement, (3) bentonite (colloidal clay), (4) Hydropel (emulsified asphalt), (5) Hydrocal (calcined gypsum), (6) Plastiment (a potential cement dispersion product), (7) silt (rock flour) and (8) fine sand (passing No. 16 sieve).

In the laboratory tests, small trial batches of the above materials in combinations, as shown in Table 1, were mixed to conform to the approximate consistency of normal rock grout. Approximate setting time, shrinkage, tendency to hold solids in suspension, resistance to water erosion and strength characteristics were observed.

The penetrating qualities of each grout were determined by pumping the grout into the interstices of medium to highly porous concrete stones. The stones were made from lightly cement-coated sand particles ranging in size from the No. 16 to No. 30 sieve. The cement-coated sand particles were molded into 1/10 cu ft cylindrical stones having approximately 30 percent voids. The stones were allowed to harden and were then used to test the penetrating qualities of the various grout mixes. In the penetration test, a stone was placed in the bottom of a cylindrical container of the same diameter as the stone and inundated with water. Grout was introduced into the bottom of the container below the stone and forced into the stone under pressure. In placing the stones in the container a seal was made between container walls and stone edges. Pressure grouting of each stone was continued until 20

psi pressure was obtained. The grout in the stones was cured for a period of 7 days, after which the stones were cut in half, and the depth of grout penetration observed.

Table 1 tabulates data for all the grout mixes tested. Mixes No. 12, 20 and 27 compare most favorably with the desired characteristics.

Mix No. 12, containing 97 percent silt and 3 percent portland cement mixed with a 3 percent bentonite and water solution, produced a grout that had high resistance to water erosion. While this grout had little or no strength, it had good penetrating qualities. This mix penetrated the porous concrete stones to a depth of 1 in. It is considered a good filler material for grouting gravel.

Mix No. 20, containing equal parts of silt and portland cement mixed with an asphaltic emulsion (Hydropel), produced grout that had excellent penetrating qualities, penetrating the concrete stones to a depth of 2 in. with an even distribution. This mix had only moderate resistance to water erosion, but hardened in 8 hours and developed approximately 60 psi strength at 24 hours. A specimen of this hardened grout was submerged in water for a period of 30 days without signs of deterioration. This mix is considered to be excellent for use in grouting gravel where imperviousness to water is desired and where low strength would be acceptable.

Mix No. 27 consisted of equal parts of silt and calcined gypsum (plaster of paris), mixed with a 3 percent bentonite-water solution. This mix attained an initial set within one hour, and at 24 hours was fairly solid, having a consistency of clay near the liquid limit. Little or no shrinkage was evident, with all solids remaining in suspension. Penetration to a depth of 2 in. was attained. While this grout had but little ultimate strength, its characteristic of becoming clayey or stiff at one hour would add to its desirability as a filler material.

The laboratory grout mixtures were developed prior to the subsurface explorations of the area in which test grouting was proposed. It was thought, prior to initiation of the program, that mixtures other than portland cement grout would be necessary to effectively penetrate zones which at that time were believed to contain considerably more sand than was later determined by exploration. As some of these grout mixtures appeared to have merit in the laboratory, the data is presented (Table 1) for what it might be worth, even though no actual field trails were made, since the portland cement grout proved effective.

The cement used in this case was Type I portland cement and had a 1600-1700 specific surface rating (Wagner). The silt used in the experimental mixes had 95 percent passing the No. 325 sieve. Fineness of other materials used in the laboratory mixtures was not determined

TABLE 1—GROUT MIXTURES

Mix No.	Solids-Parts by weight = 1.00							Water-Admix. solution = Parts by wt. of solids			Remarks§
	Silt	Portland cement	Lumnite cement	Fine sand	Hydrocal*	Plastiment†	Aerosol	Water	Bentonite	Hydropel‡	
1		0.99				0.01		0.60			Stiff at 8 hrs., hard at 24, 10 percent shrinkage
2		0.99				0.01		0.66			Set at 24 hrs., 70 percent settlement
3		0.99					0.01	0.60			3 percent settlement at 3 hrs., 15 percent settlement at 24 hrs.
4			1.00					1.52			Stiff at 2 hrs., hard at 24 hrs.
5			1.00					1.13	0.04	0.20	Final set at 5 hrs.
6			1.00					1.52	0.05		Stiff at 5 hrs., hard at 24 hrs.
7			1.00					1.29		0.23	Stiff at 5 hrs., hard at 24 hrs.
8			1.00					1.00	0.02		Final set at 3 hrs.
9		0.70	0.30					0.53		0.09	Stiff at 1½ hrs., set at 6 hrs.
10		0.69	0.30			0.01		0.66			Flash set, 50 percent shrinkage
11		0.50	0.50					1.13	0.04	0.20	No set at 8 hrs., hard at 24 hrs.
12	0.97	0.03						1.45	0.04		Stiff but not set at 24 hrs., high resistance to water erosion, good mix
13	0.94	0.03			0.03			1.41	0.04		Plastic at 24 hrs., not practicable
14	0.97	0.03						1.27	0.04	0.22	Plastic at 24 hrs., not practicable
15	0.70	0.03						0.47		0.15	Final set at 24 hrs., 10 percent shrinkage

TABLE 1 (Cont'd)—GROUT MIXTURES

Mix No.	Solids-Parts by weight = 1.00							Water-Admix. solution = Parts by wt. of solids			Remarks§
	Silt	Portland cement	Lumnite cement	Fine sand	Hydrocal*	Plastiment†	Aerosol	Water	Bentonite	Hydropel‡	
16	0.60	0.40						0.47		0.15	Stiff at 8 hrs., final set 24 hrs., 10 percent shrinkage
17	0.50	0.50						0.47		0.15	Stiff at 8 hrs., final set 24 hrs., 10 percent shrinkage
18	0.50	0.50						0.50		0.12	Plastic at 8 hrs., set at 24 hrs., 20 percent shrinkage
19	0.30	0.70						0.53		0.09	Full set 8 hrs., 10 percent shrinkage, fair mix (See Mix 20)
20	0.50	0.50						0.53		0.09	Set at 8 hrs., min. shrinkage, 70 psi 24 hrs.
21	0.50	0.50						0.85	0.03	0.28	Soft at 24 hrs., not practicable
22	0.50	0.50						0.93	0.03	0.17	No set, too wet, not practicable
23	0.50	0.49				0.01		0.50			Firm at 24 hrs., 10 percent shrinkage
24	0.97				0.03			1.27	0.04	0.22	Soft at 24 hrs., not practicable
25	0.80				0.20			1.49			Full set 1 hr., 50 percent shrinkage
26	0.50				0.50			1.49			Full set 1 hr., 50 percent shrinkage, not practicable
27	0.50				0.50			1.45	0.04		Initial set 1 hr., still plastic at 3 days
28		0.50		0.50				1.13	0.04	0.20	No set at 24 hrs., sand stays in suspension
29		0.50		0.50¶				1.13	0.04	0.20	Same as above
30		0.25		0.74		0.01		0.50			10 percent shrinkage and no set at 24 hrs.

* Calcined gypsum.
† A patended dispersion product.
‡ Emulsified asphalt.
§ All setting under water.
¶ Coarse sand No. 8—No. 16.

with the exception of the fine sand which had 100 percent passing the No. 16 sieve.

DRILLING HOLES FOR GROUTING

It was necessary to bench out a level working area in the right bank near the dam axis (See Fig. 1). The working bench was excavated through slope wash into undisturbed openwork gravel that underlies the till and was leveled at Elevation 795. An estimated 4000 cu yd were excavated in the benching operation.

A 6-in. churn drill hole (No. 61) was drilled to bedrock to determine the types of material which would be encountered in this area. Bedrock was encountered at a depth of 60.9 ft (Elevation 738.7). A log of the drill hole is shown in Fig. 1. Materials encountered consisted generally of the highly pervious openwork gravel formation encountered in previous exploration work and the less pervious gravels containing varying amounts of silt and sand, with the exception of the first 15 ft which contained a considerable number of large boulders.

It was planned to drill a number of holes on a 15 ft diameter circle by jetting methods. The number and spacing would be varied by the results obtained from grouting the initial hole (See Fig. 1).

The openwork gravel contained a narrow range of pebble sizes without sand in the interstices. Because of the absence of sand, the pebbles were in rock to rock contact, thus having a high percentage of voids, and consequently, a high permeability. In this type of material, it was hoped that the stream of jetting water would rearrange the gravel particles enough to allow the passage of a small jet pipe.

For the less pervious materials, the well graded sandy gravel, sand and the openwork gravel with sand and clay in the voids, it was assumed that a large percentage of the jet water would return to the surface through the disturbed zone around the jet pipe. As the path alongside the jet pipe had no definite dimension, it became plugged first by coarse material and then progressively by finer material until eventually the path would contain a filter allowing only the finest material to pass. It was recognized, therefore, that a path must be provided for the maximum size particle that the jetting stream would be capable of bringing to the surface.

JETTING

Several weeks were spent attempting to jet pipes into the gravel but all attempts met with complete failure. Jet pipes from ¾ to 4 in. in diameter were used with various pumps delivering up to 450 gal. of water per minute with pressures up to 100 psi. Jetting was attempted inside and outside of casings with air pressure in combination with water

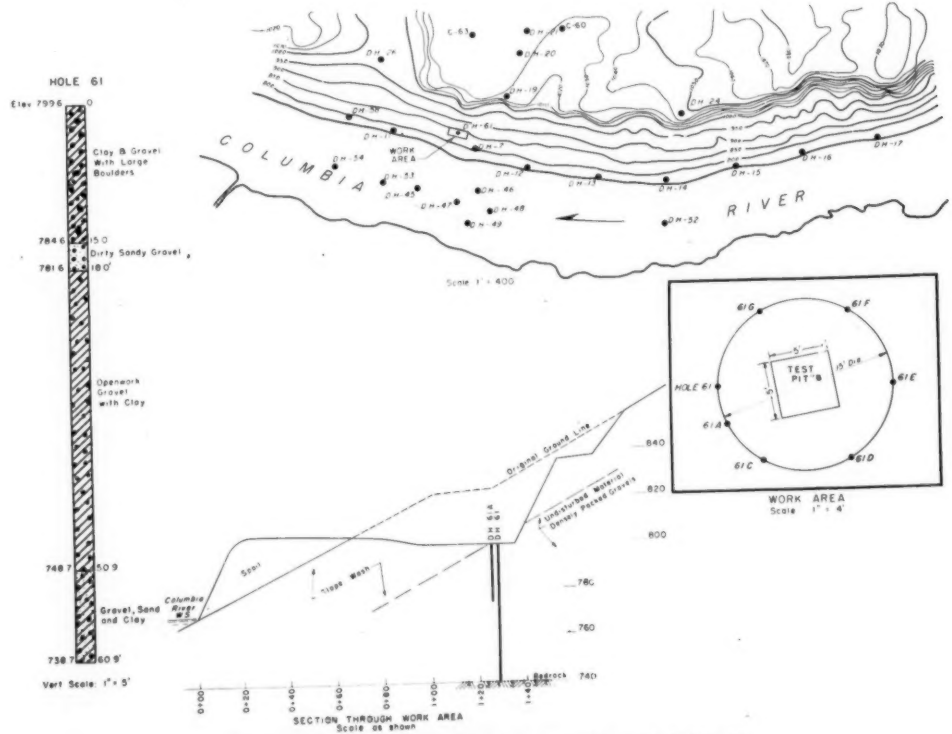

Fig. 1—Experimental grouting work area and log of hole No. 61

pressure, but without success. Rocks up to 2 in. in diameter were washed out of the hole, but the coarser gravel evidently became concentrated under the jet pipe and made progress impossible. Casings and jet pipes were vibrated with an electric vibrator and weighted with a bulldozer blade but to no avail. The boulder stratum was then penetrated by a churn drill and jetting attempts resumed in the underlying gravel, also without success.

Results of the various jetting methods used indicate that jetting is impractical in this type of material because of cobbles which concentrate at the bottom of the hole and block any working of favorable ground that might exist below.

The attempt to develop a jetting technique was abandoned and subsequent grout holes were drilled to bedrock with a churn drill, following the pattern shown in Fig. 1. Prior to pulling the casing in each hole, a 1½-in. pipe was inserted and backfilled with sandy gravel.

GROUTING EQUIPMENT AND PROCEDURE

The initial grouting was accomplished by using a Gardner Denver AF-6 valve-pot type duplex, 5¼ x 3½ x 5 in. pump fed by two combination mixer-agitator tanks. After it was determined that considerable quantities of grout would be required for the experiment, higher capacity equipment was obtained and used for the remainder of the work. This consisted of two horizontal mixers, with a capacity of 8 cu ft and equipped with two Gardner Denver AG-6 valve-pot type duplex pumps, 6 x 4 x 6 in. (Fig. 2).

Initial grouting was started in hole 61-A (Fig. 1) using mix No. 20 (Table 1) containing equal parts of silt and portland cement mixed with an asphalt emulsion (Hydropel). Considerable difficulty was immediately encountered in obtaining a mix which was comparable in

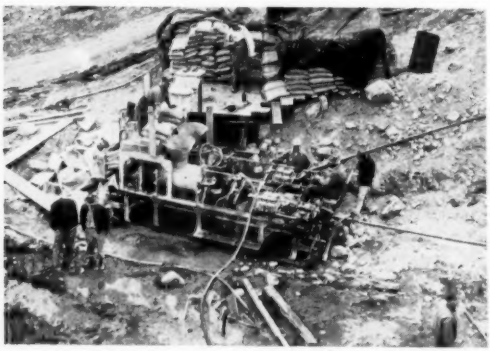

Fig. 2—Grouting equipment included double horizontal mixers with two Gardner Denver duplex pumps, type AG-6

consistency to that obtained under laboratory conditions. This condition was attributed to the effect of the low ambient temperature (10 to 20 F) on the asphaltic emulsion. Therefore, in view of the effect which the low temperatures had on the asphalt grout and the difficulty of mixing silt and portland cement under field conditions, it was decided to try a straight portland cement grout.

An initial portland cement grout consisting of one part of cement to five parts water (by volume) was prepared and pumped into hole 61-A at the 29 ft depth. This hole had been churn drilled to a depth of 24 ft and the 6-in. casing left in place. A 4-in. jet pipe had been worked down inside the 6-in. casing to a depth of 29 ft. Eleven cu ft of the grout mixture was pumped with no pressure. The grout mixture was then thickened in consistency in decrements of one part of water until 150

TABLE 2—EXPERIMENTAL GROUTING, HOLE NO. 61

Depth in hole, ft	Mix by vol.	Cement, sacks	Grout, cu ft	Max. pressure, psi	Remarks
56	1–5	3	17	0	Zone open to thin grout, but sealed with thick grout at high pressure
	1–4	1	5	0	
	1–3	2	7	0	
	1–2½	2	6	0	
	1–2	3	7	5	
	1–1½	9	18	20	
	1–1	8	12	40	
	1–0.8	2	3	150	
46	1–5	1	6	0	Zone open, grouting discontinued
	1–1½	4	8	5	
	1–1	6	9	20	
37	1–5	3	17	0	Zone open, grouting discontinued
	1–4	2	9	0	
	1–3	2	7	0	
	1–2	3	8	0	
	1–1	10	15	20	
	1–0.8	20	26	75	
29*	1–5	2	11	0	Open to thin grout, zone grouted to refusal
	1–4	3	14	0	
	1–3	6	21	0	
	1–2	6	15	0	
	1–1½	4	8	10	
	1–1	5	8	50	
	1–0.8	5	7	150	
27	1–5	2	11	0	Material in this zone tight, Grout flowed up around casing
	1–4	2	9	0	
	1–0.8	3	4	0	
Totals		119	288		

*This zone grouted in hole No. 61-A which is 3 ft from hole 61.

psi pressure was built up (See Table 2). Since it was apparent that portland cement grout would penetrate the gravel formation readily, use of other grout mixes was not given further consideration.

Grouting of hole No. 61 was begun immediately following the grouting of the 29-ft zone in hole No. 61-A. This hole was grouted in 10-ft stages from the bottom depth of 56 ft up to the 27 ft depth. The gravels surrounding this hole appeared to be highly pervious and took grout readily up to the 27 ft depth. A total of 201 cu ft of grout was pumped (Table 2) with a maximum pressure of 150 psi.

Since the gravel formation took grout readily, it was decided that the grouting stages should be decreased to 5-ft lifts to insure against ungrouted pockets and that the spacing of the holes could be increased.

Upon completion of churn drilling a grout hole to bedrock, a 1½-in. pipe was inserted within the 6-in. casing to bedrock. The well graded sandy gravel was then backfilled around the 1½-in. pipe and the 6-in. casing was pulled. The bottom of the hole was then subjected to water pressure to insure free passage of grout. Grouting was started and the mixture gradually thickened in consistency until pressure developed. Impregnation of the zone with the thickest practicable grout was continued in most cases until a maximum of 150 psi was developed. The grout pipe was then withdrawn 5 ft and the procedure repeated.

The well graded sandy gravel used for backfill around the 1½-in. grout pipes had a permeability of 2700 ft per year. It appears that the gradation of this gravel is such that it restricts the movement of cement grout through it to approximately 1 ft, therefore, the grout moved up the grout pipe only a short distance. However, the narrow wall of this gravel between the 1½-in. grout pipe and the open gravel beds at the wall of the 6-in. churn drill hole evidently offered practically no resistance to the horizontal flow of grout, probably because of washing with water prior to grouting at each horizon.

Each of the grout holes No. 61, No. 61-G and 61-F were drilled and then grouted from the bottom up in 5-ft stages to the 20 ft depth. After completion of the drilling in holes No. 61-C, 61-E and 61-G they were grouted in 5-ft stages alternating from one hole to the other from bedrock to the 20 ft depth. Very little grouting was attempted above the 25 ft depth (Elevation 770).

Grout penetrated readily through the openwork gravel and cracks in the clay. This was evidenced by the observation of grout particles in the choppings along with pH tests of water contained in samples taken from the drilling of holes No. 61-D and 61-F which was performed subsequent to the grouting of hole No. 61-A and hole No. 61. Grout was also evident in some zones in holes No. 61-C, 61-E and 61-G which were drilled subsequent to grouting of holes No. 61-A, 61 and 61-F.

The estimated cost of the grouting procedure was as follows:

Procedure	Total cost, dollars	Linear ft	Cost per linear ft, dollars
Drilling 6 in. dia. holes and placing grout pipes	5759	403	14.30
Grouting in 5-ft stages	7253	240*	30.00
Total cost			44.30

*Grouting was attempted in six holes in a 40-ft stratum above bedrock.

The grouted area surrounding the six holes required 1215 sacks per linear ft of hole. Assuming a basis of $30 per linear ft, the cost per sack for the grouting operation was $6. The experimental nature of the work, coupled with the relatively small-scale operation, resulted in this high unit cost. The actual grouting was comparable to normal foundation grouting, and it is reasonable to assume that the current costs of the latter will be applicable to this type of work at Chief Joseph Dam.

The cost of drilling the 6-in. holes and placing the grout pipes is likewise considerably higher than normal construction prices for large jobs. It is believed that in large-scale operations and by using a 4-in. drill hole, the cost of this operation would be about half of the $14.30 per ft stated above.

TEST PIT EXCAVATION AND OBSERVATIONS

A 5 ft square test pit located in the center of the circle of grout holes was excavated through the grouted area to bedrock at Elevation 748. The depth of the pit was approximately 52 ft. A horizontal drift to the grout pipe at hole No. 61-E was excavated at Elevation 775 to observe the grout in the vicinity of the pipe. A log of the pit showing the types of material and the presence of grout encountered is shown in Fig. 3.

The material in the zone of 0 to 10 ft graded from sandy gravel and boulders in the upper zone to openwork gravel and boulders with sandy clay as a matrix. This sandy clay matrix showed numerous cracks up to ⅛ in. wide between the matrix and the rock. Smaller cracks were also evident in the sandy clay itself. The size and number of cracks increased as the sand content in the clay decreased. A trace of grouted material was first observed on the south and west face of the excavation at a depth of approximately 5 ft. The remainder of the zone showed very little grout. A thin sand lens running horizontally through the pit showed no grout penetration.

From 10 to 17 ft, the excavated material consisted of gravel and boulders with clay matrix similar to that in the above zone, and was

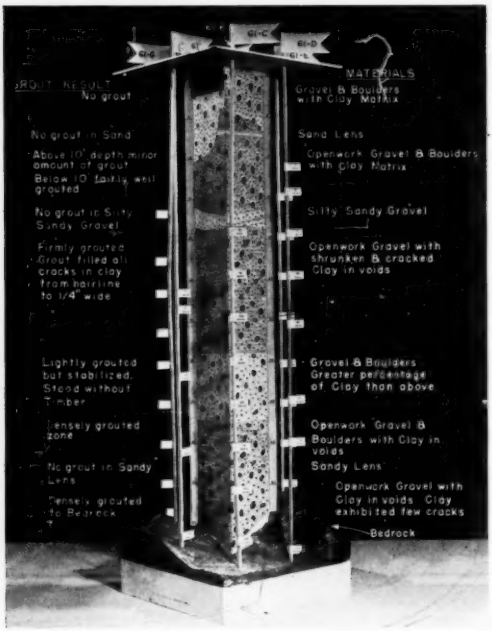
GI-C
GI-D
GI-G
GI-L
GROUT RESULTS
No grout
MATERIALS
Gravel & Boulders with Clay Matrix
No grout in Sand
Above 10' depth minor amount of grout Below 10' fairly well grouted
Sand Lens
Openwork Gravel & Boulders with Clay Matrix
No grout in Silty Sandy Gravel
Silty Sandy Gravel
Firmly grouted. Grout filled all cracks in clay from hairline to 1/4" wide
Openwork Gravel with shrunken & cracked Clay in voids
Lightly grouted but stabilized. Stood without Timber
Gravel & Boulders Greater percentage of Clay than above
Densely grouted zone
Openwork Gravel & Boulders with Clay in voids
No grout in Sandy Lens
Sandy Lens
Openwork Gravel with Clay in voids Clay exhibited few cracks
Densely grouted to Bedrock
Bedrock

Fig. 3—Model of test pit and grout holes

fairly well grouted and had been stabilized sufficiently to stand without timbering. Grout in this area appeared in practically all of the cracks and between the rocks and clay and even the cracks 1/100 in. in width were filled with grout. (See Fig. 4)

A lens of silty sandy gravel approximately 2 ft thick was encountered at depth of approximately 17 ft. This stratum contained no grout and allowed seepage of water into the excavation. With the rise in the river to Elevation 782, water flowed through this lens at the rate of approximately 120 gpm. A concrete collar was placed around the pit at this depth, effecting a seal and stopping the flow of the water. Laboratory tests on material from this stratum gave a permeability of about 50,000 ft per year; however, this increased to 1,500,000 ft per year on samples obtained from the face after water had been flowing for several days and had washed out the fines. Considering this permeability and the low head of only a few feet, it is evident that the river water was approaching within about 10 ft of the pit in some form of an open channel, possibly through a channel that had been washed out during the jetting operation on hole 61-A. It was noted that the water was all coming through the

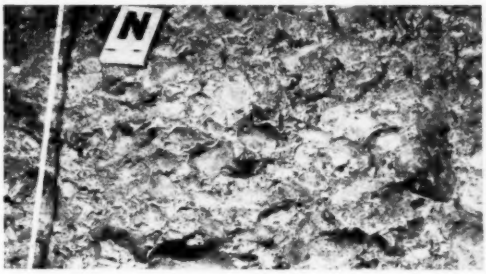

Fig. 4—Gravel formation after grouting

pit in the vicinity of holes 61 and 61-F. As very little grouting had been attempted above the 25-ft depth, it is evident that effective grouting of the channel was not accomplished.

The zone between 19 and 34 ft consisted of openwork gravel with shrunken, cracked clay in the voids. The material was firmly grouted and stood without timbering. Excavation through this area and in the remainder of the pit was facilitated by blasting. The blasting operation was conducted with extreme care to avoid disturbance of the surrounding grouted areas.

Between 34 and 42 ft, the material consisted of gravel and boulders with a greater percentage of clay than the zone above. While the zone did not appear to be densely grouted, it was nevertheless stabilized and stood without timbering. No appreciable leakage of water into the pit through this area was evident.

From 42 ft to bedrock, the material excavated was openwork gravel and boulders, firmly grouted, and had the appearance of concrete. Between 47 and 48 ft, a fairly stable sandy lens was encountered in which little grout appeared. There was very little leakage of water into the excavation through this lens.

The only sand lenses encountered were at depths of 6, 17 and 47 ft, and these varied in thickness from a few inches to approximately 2 ft. The sandy material was classified as coarse sand, being fairly well graded to the No. 50 sieve with only a minimum percentage passing the No. 50. Little or no grout penetrated into the sandy lenses.

At the completion of the excavation, at Elevation 749, the inflow of water into the pit was approximately 12 gpm (river elevation 777 ft). Partial drawdown tests verified the assumption that the bulk of this flow was coming from the upper stratum which contained more sand and was not firmly grouted.

The bottom of the pit reached bedrock in the northeast corner at a depth of approximately 51 ft, and the rock dropped off sharply towards

the southwest. Bedrock was clean irregular granite, well rounded by water action. When the deepest corner was excavated, the inflow inincreased rapidly to about 80 gpm and was evidently coming through a fissure in the bedrock where grouting had not been effective. The leak was successfully sealed by grouting after the water had stabilized in the pit.

The horizontal drift to the grout pipe at hole 61-E was excavated at Elevation 775. The grout pumped into this hole contained an organic dye (blue) and its course of penetration was easily recognized. As shown in Fig. 5, the grout formed a solid column up and around the full length of the pipe exposed and did not penetrate the undisturbed gravel away from the pipe. This condition would indicate that the surrounding gravels had been grouted from some other hole on the circle leaving only a narrow path along the pipe in which the grout was confined. This hypothesis is further confirmed by a study of the grouting operations of hole 61-E, Table 3, in which it is shown that the line above the 30 ft depth was tight and took comparatively little grout.

The quantities of grout pumped in holes 61-G, 61-E and 61-C, were considerably less than the alternate holes 61-F, 61-D, and 61 which were grouted subsequent to the drilling of holes 61-G, 61-E, and 61-C. This would indicate that grout will penetrate this formation in excess of 10 ft.

The effectiveness of grouting of this formation is shown in Table 4, by pumping tests, which were run in other test pits excavated in this formation for exploration prior to the grouting experiments.

SUMMARY AND CONCLUSION

Information on the formation of materials prior to the grouting experiment indicated a need for a grout which would have higher pene-

Fig. 5—Grouted area around hole No. 61-E at 25 ft depth

TABLE 3—EXPERIMENTAL GROUTING, HOLE NO. 61-E

Depth in hole, ft	Mix by vol.	Cement, sacks	Grout, cu ft	Pumping time, min.	Max. pressure, psi	Remarks
51	1–2	10	25	25	180	Zone sealed
46	1–3	2	7	5	150	Tight, zone sealed
41	1–3	6	21	15	25	Open work gravel, could not build up pressure, grouting discontinued
	1–2	6	15	5	25	
	1–1½	2	4	5	25	
	1–1	6	9	10	25	
	1–0.75	16	20	—	20	
36	1–3	5	18	—	90	Tight material, zone sealed at maximum pressure
	1–2	4	10	—	90	
	1–1	6	9	—	100	
	1–0.75	8	10	—	150	
30	1–2	10	25	—	150	Zone sealed
25	1–3	4	14	15	70	Dense material, zone sealed
	1–2	8	20	40	120	
	1–1	9	14	20	210	
20	1–3	3	11	15	160	Hole sealed
15	1–3	4	14	10	60	Grout came to surface about 3 ft from pipe toward center of circle
Totals		109	246			

Note: Organic dye (methyl blue) added to grout.

TABLE 4—PUMPING TESTS

Test pit No.	Location	Elev., ft	Drawdown, ft	Pumping rate, gpm
2	400 ft upstream from grout area, 100 ft from river	825 to 752	4.5	450
2A	Excavated at the end of exploration tunnel 400 ft upstream from grout area and 1200 ft from the river into the abutment	800 to 769	0.3	160
8	Grouted area	795 to 748	26	8

trating qualities than portland cement grout. Accordingly several grout mixes were developed in the laboratory which would meet this requirement. However, it was found that grout mixes containing asphalt emulsion are not practical in extremely cold weather. Since in the initial trial the portland cement grout appeared to be penetrating the formation successfully, no further consideration was given to use of other grout mixes developed in the laboratory. Further field tests will be required to demonstrate the value of laboratory grout mixes for field use.

Drilling of holes by jetting methods results in an accumulation of boulders at the base of the jet pipe and it is evident that as the fine material is washed to the surface, the larger particles cave into the crater from the sides making further penetration impossible. Drilling of grout holes in this formation required the use of a heavy churn drill rig and progress was limited to about 10 ft per shift.

The use of a well graded sandy gravel ($-\frac{3}{4}$ in.) for back-filling drill holes around the grout pipes is a very effective method of preventing grout from flowing up the outside of the grout pipe casing.

Observations in the test pit showed that the gravel was well sealed and effectively grouted from a depth of 23 ft (Elevation 772) to 52 ft (Elevation 743) where bedrock was encountered. Areas of excessive leakage occurred in the upper zone where very little attempt had been made to grout and at bedrock where cracks and fissures were not sealed.

The following is considered an effective procedure: (1) grouting from bedrock up, using 5-ft stages; (2) alternating between holes, and (3) using an initial mix of one part cement to 5 parts water with gradual thickening until a pressure of 150 psi is obtained.

A spacing of grout holes 7.5 ft apart will produce an effective seal and it is indicated that an increased spacing can be used.

ACKNOWLEDGMENT

The experiment described herein was made at the Chief Joseph Dam site and in the Seattle District Laboratory under the direction of Col. L. H. Hewitt, District Engineer, Corps of Engineers, Seattle District. The text, tables, and figures used in this paper were taken from a report prepared by the Seattle District. The program was planned and directed by G. B. Hunt. The laboratory testing and grouting was done under the supervision of the writer. The field work was under the supervision of R. L. Nichols. Byron W. Steele and V. L. Minear of the Office, Chief of Engineers, Washington, D. C., gave valuable assistance in the planning of the tests and made many valuable suggestions during the execution of the work.

Title No. 46-24

Significant information on pozzolans—particularly fly ash and treated diatomite

Use of Pozzolans in Concrete*

By RAYMOND E. DAVIS†

SYNOPSIS

After stating the characteristics of pozzolanic materials, the effects of replacing with pozzolans part of the portland cement in concrete are considered briefly. Results of tests with fly ash and superfine diatomite are cited to show the possible use of these materials in the East and Midwest where natural pozzolans of the West are not economically available.

INTRODUCTION

Until the development of natural cements less than two centuries ago, the only hydraulic cements were those composed of a mixture of pozzolan and lime. Such cements certainly date back to ancient Rome and Greece when they were extensively used in all types of masonry construction.

In more recent times materials possessing pozzolanic properties have been used as an ingredient of portland cement concrete, either added at the mixer or blended or interground with the cement. Their use in this manner has been considerably more extensive in Europe than in the United States although their use in this country is increasing.

A pozzolan may be defined as a siliceous material which, while in itself posessing no cementitious value, will in finely divided form and in the presence of moisture react chemically with calcium hydroxide at ordinary temperatures to form insoluble compounds possessing cementitious properties.‡ There is much that we do not know concerning the chemistry of pozzolanic reaction. In this respect our position is about the same as it was 25 years ago with respect to portland cement.

However, we do know that the rate of reaction is dependent upon both the mineral composition and fineness of the pozzolan. In this regard there is a great difference between pozzolans, and one of our important

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‡Davis, Raymond E., Carlson, R. W., Kelly, J. W. and Troxell, G. E., "Properties of Mortars and Concretes Containing High-Silica Cements," ACI JOURNAL, Mar.-Apr. 1934, *Proc.* V. 30, p. 369.

jobs is to be able to distinguish between those which may suitably be employed as additions or as cement replacements, and those which should not be so used. Though there are a number of short-time tests for evaluating the ability of a pozzolan to combine with calcium hydroxide, its effect upon the properties of hardened concrete can be determined with certainty only by long-time tests.

COMPOSITION OF POZZOLANS

While the principal constituent of all pozzolans is silica, some pozzolans exhibiting superior performance in concrete contain 40 percent or less of this compound. Most pozzolans contain substantial quantities of alumina and iron oxide, and it seems that all good pozzolans contain at least a small quantity of the alkalies.

Silica in the amorphous forms reacts with calcium hydroxide liberated during the hydration of portland cement much more rapidly than does silica in the crystalline forms. Thus we may separate siliceous materials into pozzolans, those in which at least a substantial proportion of the silica content is of amorphous structure, and non-pozzolans, those in which the silica constituent is of crystalline structure.

Among natural pozzolans it appears that those which are most active are high in opal; these include the diatomites and opaline cherts and shales. Less active are the volcanic glasses such as the pumicites and tuffs. The pozzolanic properties of many materials, including some of the clays which are unreactive in their natural state, may be improved by suitable calcination or heat treatment.

Perhaps the most important of the artificial pozzolans in this country are the fly ashes. Fly ash is the finely divided residue from powdered coal which is caught in electric precipitators in steam power plants. Many of the glassy particles of which fly ash is composed are smooth and nearly spherical in shape. Depending upon the coal and power plant conditions, the carbon content of American fly ashes ranges from less than one to more than 20 percent, and their silica content ranges from less than 30 to more than 45 percent. Some of them are considerably finer than normal portland cement.

CEMENT REPLACEMENT

The optimum amount of a pozzolan which may be used depends upon the properties of the concrete which it is desired to enhance, the character and fineness of the pozzolan, the richness of the mix and other factors.* As a cement replacement, this optimum amount may be as

*Davis, Raymond E., Kelly, J. W., Troxell, G. E. and Davis, Harmer E., "Properties of Mortars and Concretes Containing Portland-Pozzolan Cements," ACI JOURNAL, Sept.-Oct. 1935, *Proc.* V. 32, p. 80.

low as 4 to 6 percent for fine ground diatomite to as high as 35 or perhaps even 50 percent for some of the fly ashes.

Generally speaking, concretes in which a pozzolan is used as a replacement for a portion of the portland cement are more plastic, bleed less, and exhibit less segregation than do corresponding concretes containing straight portland cement. For finely divided pozzolans of opaline character, these beneficial effects may be large, particularly for wet mixes and for concretes of low cement content.

For equal slumps, the water requirement of concrete is generally greater when a pozzolan is used than when a straight portland cement is employed. However, slump is not a valid criterion in comparing the workability or placeability of concretes containing pozzolans with that of corresponding concretes containing straight portland cement. Some of the pozzolans greatly increase the "fatness" of concrete, and it is a matter of common observation that such a concrete of 1- or 2-in. slump may be just as readily placed as a corresponding straight portland cement concrete of considerably higher slump. The Powers remolding apparatus provides a much more reasonable measure of workability of pozzolan-cement concretes than does the slump test.

At least for certain of the pozzolans, particularly those of very high fineness, the use of a suitable air-entraining agent results in a water cut substantially greater than that obtained when the same agent is employed in concrete containing straight portland cement.

At the early ages, concretes containing pozzolan in the normal amounts of cement replacement generally exhibit compressive strengths which are appreciably lower than those of corresponding concretes containing normal portland cement without replacement. At the later ages, pozzolan-cement concretes generally exhibit compressive strengths which approach those of corresponding concretes containing normal portland cement. The use of pozzolan as a cement replacement generally enhances strength more in lean or harsh mixes than in rich mixes.

Perhaps one of the most important characteristics of pozzolan-cement concretes is their high degree of watertightness. At the later ages the permeability coefficient of such concretes is likely to be only a fraction of that of corresponding concretes containing straight portland cement.

In amount depending upon the pozzolan employed and the percentage of the replacement, the heat of hydration of a portland-pozzolan cement will be less than that of a corresponding straight portland cement.

Concretes containing a suitable pozzolan in appropriate amount are substantially more resistant to sulfate attack than are corresponding concretes containing Type I, Type II or Type III cements. They are also substantially more resistant to weakly acid or low pH waters.

In general, without purposeful air entrainment, pozzolan-cement concretes may be expected to offer somewhat less resistance to weathering than do corresponding concretes containing normal portland cement, though the results of freezing and thawing tests conducted in the laboratory indicate that the reverse may be true for certain pozzolans when used in the normal amounts of replacement, including some of the fly ashes and finely ground diatomites. However, when some of the air-entraining agents are employed it has been found that in some instances pozzolan-cement concretes are far more resistant to freezing and thawing than are the corresponding air-entraining straight portland cement concretes. This has been attributed to the relatively high degree of impermeability exhibited by the pozzolan-cement concretes.

Generally speaking, pozzolan-cement concretes exhibit a greater drying shrinkage than do corresponding concretes containing normal portland cement. Some of the fly ashes of low carbon content and high fineness, when used as cement replacements up to 30 percent, produce concretes for which the drying shrinkage is no greater than that of a corresponding concrete containing straight portland cement. Certain air-entraining agents, which substantially reduce the water requirement, act to reduce the drying shrinkage of some of the pozzolan-cement concretes to a value less than that exhibited by a corresponding concrete containing straight portland cement without air-entraining agent.

Many of the pozzolans, when used in appropriate amounts, are very effective in preventing the expansion accompanying alkali-aggregate reaction. Other things being equal, those which are high in opal, such as the diatomites and opaline shales, are considerably more effective in counteracting excessive expansion than those which are high in glass, such as the pumicites, tuffs and fly ashes.

APPLICATION TO CONCRETE CONSTRUCTION

If the previous statements are accepted, then it would appear that, with a suitable air-entraining agent which would cut drying shrinkage and promote weathering resistance, a suitable pozzolan-cement mixture could be appropriately used for any type of concrete construction, regardless of conditions of exposure. In the construction of buildings, disregarding the sometimes favorable item of cost, the chief arguments for the use of a pozzolanic material as a cement replacement instead of straight Type I portland cement would generally be greater workability, less tendency towards segregation and bleeding, and exterior walls and floors less susceptible to the passage of moisture. For such general concrete construction the cost of materials and labor is always likely to be the item of primary consideration. If it can be shown that the total cost would be less when pozzolan and an air-entraining agent

are employed, then these materials will be used in preference to a straight portland cement as an economy measure.

One of the bad features of a concrete pavement is that the top is where the bottom ought to be. While air entrainment tends to reduce bleeding, it does not prevent segregation of the larger aggregate sizes. Even for the dry mixes customarily employed in pavement construction, the quantity of coarse aggregate in the top inch of the slab is likely to be substantially less than in the bottom inch. The potential shrinkage of the top inch of a slab may be two or three times as great as that of the bottom inch. It has been observed that a small percentage of some of the pozzolans is many times as effective as entrained air in reducing segregation, preventing bleeding, and producing a concrete slab which is homogeneous from top to bottom. Through the use of a suitable air-entraining agent and a suitable pozzolan it may be possible to construct pavements having low drying shrinkage and high resistance to weathering, and at the same time provide a homogeneous concrete throughout the depth of the slab.

Pozzolans seem to fit chiefly into the construction picture in mass structures such as dams and bridge piers, where low heat of hydration is desired; in hydraulic structures of all kinds, where watertightness is important; in structures subject to alkali or acid attack from ground water, sea water, or from industrial wastes; and in underwater construction where concrete is deposited by tremie or bucket.

For the very lean air-entraining mixes, used in dams, replacement of a portion of the portland cement by a suitable pozzolan appears to be highly desirable, not only to decrease the permeability of the hardened concrete but also particularly to improve workability and reduce bleeding and segregation.

FLY ASH

The use of fly ash as a pozzolan added at the mixer is becoming extensive in some of the larger cities of the East and Middle West. Generally speaking, where fly ash is used as a cement replacement up to 30 percent by weight, the lower its carbon content and the higher its fineness the lower will be the water requirement of fresh concrete, the higher will be the compressive strength at all ages, and the lower will be the drying shrinkage.* The carbon content appears to have little effect upon the weathering resistance of concrete in which fly ash is employed.

As illustrative of the effect of carbon content upon properties of concrete, when used in a six-sack mix as a 20 percent by weight replace-

*Davis, Raymond E., Davis, Harmer E. and Kelly, J. W., "Weathering Resistance of Concretes Containing Fly Ash Cements," ACI JOURNAL, Jan. 1941, *Proc.* V. 37, p. 281.
Davis, Raymond E., Carlson, Roy W., Kelly, J. W. and Davis, Harmer E., "Properties of Cements and Concretes Containing Fly Ash," ACI JOURNAL, May-June 1937, *Proc.* V. 33, p. 577.

ment of the cement, a low carbon fly ash *decreased* the water requirement three points while a high carbon fly ash *increased* it six points.

Using low carbon fly ash as a 20 percent cement replacement, the compressive strength of concrete was 300 psi lower at the age of 7 days, but 800 psi higher at the age of one year than that of a corresponding concrete containing Type II cement. At the other extreme, for a coarse fly ash of very high carbon content the compressive strength of concrete at the age of 7 days was 1200 psi lower and at the age of 1 year 1100 psi lower than that of a concrete containing straight Type II cement.

Carbon content and fineness also affect the drying shrinkage. For a 20 percent replacement of cement by a low carbon fly ash, the 1-year drying shrinkage of concrete was 7 percent *less* and for a high carbon fly ash was 25 percent *greater* than that of a corresponding concrete containing straight Type II cement.

For a 4-sack concrete at the age of 6 months the permeability coefficient using fly ash as a 30 percent replacement was only 1/5 as great and as a 50 percent replacement was only 1/12 as great as that for a corresponding concrete containing portland cement without fly ash.

Freezing and thawing tests were made on concretes containing fly ashes as 10 and 20 percent replacements, for which the carbon content varied from 1 to 17 percent. Regardless of the carbon content, the resistance was higher for concretes containing fly ashes than for the corresponding concretes containing straight portland cement. Those of high carbon content were just as satisfactory as those of low carbon content, and quite generally those containing fly ash as a 20 percent cement replacement exhibited a weathering resistance superior to those containing 10 percent replacements. It is significant that, on the average, the concretes containing 20 percent replacements exhibited a resistance to freezing and thawing, as measured by a 25 percent loss in weight, which was approximately double that of the corresponding concrete containing portland cement without replacement.

From the above it seems evident that fly ashes of high fineness and low carbon content are excellent pozzolans suitable for general concrete construction. When employed as cement replacements in moderate amount, say up to 25 or 30 percent, fly ash-cement concretes will possess properties of strength, drying shrinkage, and weathering resistance approximately equal to and in some cases slightly superior to corresponding properties of concrete containing straight portland cement. The use of the coarse fly ashes of high carbon content is undesirable except with respect to weathering resistance.

The fly ashes are not the equal of the finely divided diatomites and opaline cherts and shales in improving the workability, decreasing the

tendency towards bleeding and segregation, and counteracting the effect of alkali-aggregate reaction.

TREATED DIATOMITE

In earlier years pulverized diatomites, such as Celite, were extensively employed as small additions to improve workability and prevent segregation of fresh concrete. However, such use in substantial amount was found to increase the water requirement and also the drying shrinkage of concrete. Recently it has been found that a small percentage of a finely ground diatomite treated with a suitable air-entraining agent not only enhances those properties of fresh concrete in which we are interested but also substantially increases the compressive strength at the early as well as the later ages, substantially reduces the shrinkage, and greatly increases the resistance to freezing and thawing.

To explore the possibilities of improving the quality of concrete through the use of small replacements of superfine diatomite and purposely entrained air, diatomite treated with certain air-entraining agents was so finely ground that all signs of the diatom structure as determined by microscopic examination were destroyed. The specific surface of this superfine material was approximately ten times as great as that of normal portland cement. When this treated material was used as a 6 percent replacement in a 5¼-sack mix the water-cement ratio for 5 percent air entrainment was approximately 6 points lower than for the corresponding straight portland cement without air entrainment and approximately the same as for the corresponding straight portland cement with air entrainment.

For this 6 percent replacement, and one of the air-entraining agents, the 7-day compressive strength was 68 percent higher and the 3-month compressive strength was 34 percent higher than that of the corresponding straight portland cement concrete without air entrainment; and the strengths at all ages for the concrete containing the treated diatomite were substantially greater than the corresponding strengths for the straight portland cement concrete containing the same agent.

After 6 months of drying, the shrinkage of concrete containing a 6 percent cement replacement of this treated diatomite was 8 percent less than that of the corresponding concrete containing straight portland cement without air-entraining agent, and 15 percent less than that of concrete containing diatomite without air-entraining agent. Of particular significance was the effect of a 6 percent superfine diatomite used as a cement replacement upon the weathering resistance of concrete. Regardless of the air-entraining agent used, the resistance to freezing and thawing was substantially greater for concretes containing superfine diatomite than for corresponding concretes containing straight portland cement and the same air-entraining agents. For one of the

agents employed, the resistance to freezing and thawing of concrete containing the treated diatomite was approximately four times as great as that of the corresponding concrete containing straight portland cement, 13 times as great as that of the straight portland cement without the air-entraining agent, and 16 times as great as that of the concrete containing superfine diatomite without the air-entraining agent.

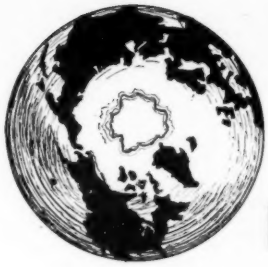
For a lean mix containing approximately 3 sacks of cement per cu yd (for which the required consistency was determined by remolding effort), the use of an agent to produce 5 percent of entrained air in straight portland cement concrete reduced the bleeding of fresh concrete by approximately one half. When treated diatomite was employed as a 6 percent cement replacement, the bleeding was 1/6 that of the straight portland cement concrete without air-entraining agent, and when employed as an 11 percent replacement, the bleeding was 1/17 that of the straight portland cement concrete wihout air-entraining agent.

Tests were made on a straight portland cement concrete without air-entraining agent for which the cement content was 5¼ sacks per cu yd, and on a treated diatomite-cement concrete, for which the portland cement content was only 4¼ sacks per cu yd, the diatomite content was 25 lb per cu yd, and the air content was 5 percent. The compressive strength of the concrete containing the treated diatomite with one less sack of cement per cu yd was 10 percent higher at the age of 7 days and 4 percent lower at the age of 3 months than that of the 5¼ sack straight portland cement concrete without air-entraining agent. The drying shrinkage at the age of 6 months was 14 percent less for the lean-mix, treated-diatomite concrete than for the richer mix portland cement concrete, and the resistance to freezing and thawing of the lean-mix, treated-diatomite concrete was six times as great as that of the richer mix, straight portland cement concrete.

These results, which are rather startling, indicate that, through the use of a small percentage of superfine diatomite treated with a suitable air-entraining agent, there is a possibility of reducing the cement content of concrete as much as a sack per cu yd without sacrifice of strength and perhaps at a lower over-all cost, at the same time substantially improving the properties of drying shrinkage and weathering resistance.

CONCLUSION

From the standpoint of economy as well as quality, suitably constituted fly ashes used in the larger replacements and superfine diatomites treated with a suitable air-entraining agent used in the smaller replacements, offer attractive possibilities in the East and Middle West, where the natural pozzolans generally found in the West and generally used in the higher percentages of replacement may not be economically available.



Letters from Readers*

BY WAY OF SYNOPSIS

S. K. Ghaswala describes an experimental prefabricated house built in India.

A test report and references on autogenous healing of concrete are supplied by H. F. Gonnerman in answer to a reader's inquiry.

Ala E. Fatin discusses the use of salt water for mixing concrete.

A reader asks about the effect of pile driving vibrations on concrete during and after placement.

Another asks for information on the corrosion of reinforcement in flat slabs with a partial answer given by the Editor.

Oivind Moe comments on the addition of calcite to cement.

E. Copeland Snelgrove discusses air entrainment, slump and grading practices used in England.

More information on soil pressures under a continuous foundation is given by M. Hirschthal.

G. L. Blanchard and J. M. Steinbrugge discuss the construction of hexagonal bins.

An Experimental Prefabricated Concrete House in India (LR 46-41)

S. K. Ghaswala, Deputy Chief Engineer, Concrete Construction and Engineering Co., Ltd., Bombay, India, describes an experimental prefab in India.

An experimental single-story two-unit structure using the structural panel system of construction is believed to be the first of its kind erected in India (Fig. 1). The house has a floor area of 705 sq ft, cubic content of 9170 cu ft and costs 11,000 rupees ($3400). It demonstrates the flexibility of modern concrete construction and affords a means for obtaining information on its permanence and strength under actual weather conditions, as well as the efficiency of hollow panels for thermal and acoustic insulation. The ultimate success of this venture will depend on good

*A part of copyrighted Journal of the American Concrete Institute, V. 21, No. 5, Jan. 1950, *Proceedings*, V. 46. Separate prints are available at 35 cents each. Address 18233 W. McNichols Rd., Detroit 19, Mich.

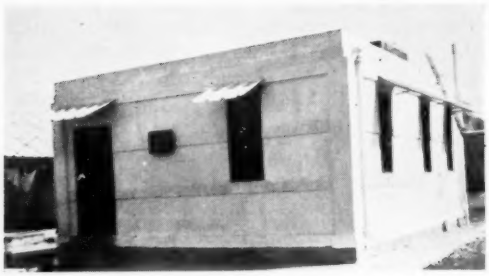


Fig. 1—Experimental prefab in India

workmanship and technique in construction, and accuracy in dimensioning and casting the units.

DESIGN AND CONSTRUCTION

In the structural-panel design adopted, the maximum weight of the components is about 400 lb, permitting the handling of the larger units by the simplest mechanical arrangements and the lighter ones by manual labor. Further, this construction offers considerable flexibility in design and layout. All precast standardized components are of vibrated concrete cast in wooden molds, giving a compressive strength considerably above that of normal concrete and permitting higher design stresses. For the floor beams, although it is possible to adopt higher working stresses, as for example, 1000 to 1200 psi in compression, the design is limited to 700 psi. The maximum length of floor beams is 11 ft 10 in. and they are reinforced with three $\frac{3}{8}$-in. mild steel bars at the bottom only.

The wall panels are hollow, precast, vibrated concrete sections, 2 ft 6 in. wide, 2 ft 8 in. high and 8 in. thick and weigh about 300 lb. Their comparatively small size allow them to be handled more easily and conveniently than the usual large precast panels. Top and bottom grooves interlock directly with adjoining splay pieces whereas the projections of the two vertical side grooves abutt against the projections of adjacent wall panels to form a hollow core 8 x 3 in. between all panels. This core is later filled with concrete reinforced with two $\frac{3}{8}$-in. mild steel bars to serve as a continuous rigid vertical member from the ground level to the second floor level, and in some cases (wherever they support floor beams cast-in-situ), up to the roof level. These vertical members are cast-in-situ on 2 ft 6 in. centers to firmly bind the wall panels. Horizontally, the structure is securely bound by splay pieces which contain longitudinal and spiral steel of $\frac{3}{8}$ in. diameter mild steel bars, the former being kept a few inches longer than the splay piece itself. These longitudinal bars are subsequently attached to the vertical reinforcement of

the columns and also with each other, thus forming a continuous horizontal beam along the whole length of the building. Both the panels and the splay piece have an external groove, ½ x 1 in., which forms a seat for plaster subsequently applied to conceal all joints.

The floor beams rest on U-shaped top splay pieces. The groove formed by the "U" is filled at the site with 1:2:4 concrete reinforced by four ⅜ in. diameter mild steel bars. To interlock the wall panels from second floor level upwards and also to rigidly bind the members at the second floor level, for structures more than one story height, a longitudinally grooved beam 8 in. wide and 3 in. deep is cast-in-situ directly above the floor beams at their periphery. These grooved beams contain reinforcement connected to that in the vertical columns. Thus the wall panels along with the cast-in-situ columns, floor beams and grooved beams act as rigid vertical members preventing lateral motion at first floor and roof levels. In view of this, wind pressure calculated at 20 lb per sq ft is resisted easily. As the total height of the structures is less than 25 ft and as they will always be grouped together, the assumption of such a wind pressure is more than justifiable in this case.

The solid L-shaped corner panels serve as a convenient form for the corner columns. They are of the same height as the wall panels, *viz.*, 2 ft 6 in., and contain ¼-in. mild steel reinforcing bars for binding with the end columns.

Lintel panels over the doors and windows are provided. These lintels are solid in section immediately below the floor beams and hollow in section above the lower frame, *i.e.*, a combination of a solid splay piece and a half hollow wall panel.

Internal partition walls are of hollow block. In walls parallel to the floor beams, a cast-in-situ beam is provided underneath and flush with the floor beams. These walls are only 7 to 8 ft high and require no support underneath. For block walls transverse to the floor beams but of the full room height no special cast-in-situ beams are necessary because these walls rest one above the other. The load from the second floor wall is transmitted to the first floor wall through the transverse floor beams.

ERECTION

Erection of the first precast structure took over a month, but with more experience and proper planning it should not take more than 10 to 12 days. Construction of the foundation of these houses is carried out as for ordinary building foundations. A small tongue is provided on the top of the foundation wall. Reinforcing bars in the foundation are joined with the horizontal reinforcement of the floor beam.

After the tongue joint is completed, the wall panels are placed on the base so that the horizontal grooves fit over the projecting tongue in the base. After the lowest panels are placed, the columns formed by the projections of the wall panels are concreted. These 8 x 3 in. columns are reinforced by the bars previously placed in position and securely embedded in the foundation. Splay pieces then are placed so that their projections fit into the longitudinal grooves of the wall panels. Above the splay pieces, another set of wall panels is placed so as to interlock with the respective grooves. A vertical set of three wall panels and three splay pieces provides the requisite height for rooms. At the corners of the rooms or at the junctions of walls, reinforced corner or tee-panels are placed.

For a window or door, two or more wall panels are left out. A special reinforced lintel, is placed above the door and window openings.

The floor beams rest on a U-shaped top splay piece which is placed on the wall panel. For wall panels which do not support two such sets of floor beams, special splay pieces are provided. Above the floor beams and running parallel with the wall panels a cast-in-situ grooved beam is laid. Reinforcing bars coming out from the columns at suitable intervals are bent into these grooved beams.

The second story is erected in the same manner as the first. The roof formed by hollow beams is placed in the same way as the floor beams, with the addition of a waterproof covering. Panels and beams were erected satisfactorily with a hand-operated stacker.

Autogenous Healing (LR 46-42)

A reader asks for information on the autogenous healing of concrete.

What references are there on the effect of breaking partially or completely hardened concrete? I understand that concrete when broken and kept clean will knit together like a living bone and possibly be even stronger than when originally tested.

H. F. Gonnerman, Assistant to Vice-President for Research and Development, Portland Cement Assn., Chicago, Ill., supplies a test report and list of references.

Duff A. Abrams made tests on the autogenous healing of concrete at the Structural Materials Research Laboratory, Lewis Institute, Chicago, in July, 1925, and his report is quoted.

The testing of 8 year old concrete is of itself sufficiently unusual to attract attention. Recent tests made at the Structural Materials Research Laboratory, Lewis Institute,

Chicago, were unique in that the 8 year old concrete specimens had been previously loaded to failure when 28 days old. After the original test, which did not shatter the concrete, the cylinders were thrown outdoors on the ground exposed to the weather.

The compressive strengths of five 6 x 12-in. cylinders, 1 part cement to 5 parts aggregate by volume were:

28-day strength, psi	2330	2510	3130	2360	1580
Strength of same retested after 8 years	5070	4200	5290	4940	5980
Ratio of 8-yr. to 28-day strength, percent (Average 228)	217	167	169	210	379

The variation in strength was due to differences in the size and grading of the aggregates and in the consistency of the concrete.

The cracks which were caused by the 28-day test closed up during 8 years exposure to weather. This may be explained by the slight solubility of both hardened cement and aggregate in rain water. Depositing of soluble compounds in the old cracks formed a good bond.

The writer has called attention to similar phenomena in the following instances:

1. During the loading of a reinforced concrete test bridge for the Illinois Highway Commission, diagonal tension cracks near the ends of the girders which had been produced 3 years before by a preliminary test load (when the bridge was 2 months old) were carefully mapped and photographed. In the final test many of the old cracks did not reopen, although the load on the bridge was about 6 times the load which caused the cracks. For a detailed report see "Test of a 40-foot Reinforced Concrete Highway Bridge," *Proc.*, A.S.T.M., 1913.

2. Evidence of autogenous healing of early injuries due to overstressing in bond or compression will be found in "Tests of Bond between Concrete and Steel," Bulletin 71, University of Illinois Engineering Experiment Station, 1913.

It is probably a new idea to most users of portland cement concrete that in the presence of moisture, small cracks will heal and in fact become much stronger than before the break; it is reassuring to know that concrete possesses this remarkable property.

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Can Salt Water Be Used Effectively for Mixing Concrete? (LR 46-43)

ALA E. FATIN, Director of Works, Port of Alexandria, Egypt, answers a question asked in the *We Want to Know* section of the October JOURNAL.

The latest research appears to have established that the use of pure sea water is not harmful[1] in the preparation of concrete, although specifications usually forbid its use. Port and lighthouse engineers in Egypt use pure sea water in mixing concrete for large plain concrete blocks, where fresh water is not available, without any noticeable ill effects.

D. A. Abrams[2] reported that concrete mixed with sea water (about 3.5 percent salts, mostly sodium chloride) and cured in the moist room gave higher strengths than fresh water concrete at ages of 3 and 7 days. At 28 days and over, the strength ratios for sea water ranged from 80 to 88 percent. Air cured concrete mixed with sea water was lower in strength at 3 months, but showed a recovery of strength at later ages and gave strengths equal to that with fresh water.

Experiments at the Egyptian State Railways Bridge Laboratories showed that the tensile and compressive stresses of 1:3 mortar mixed with sea water exceeded slightly those mixed with fresh water.

In spite of the satisfactory strength results, J. Singleton-Green[3] stated that it seems unwise to use sea water in reinforced concrete constructions particularly in the tropics, because of the danger of corrosion of reinforcement. Results of experiments carried out by J. S. Swell[4] also supported this opinion.

The strength of aluminous cements is affected adversely[5] by the presence of calcium chloride in the mixing water. Though sea water has often been used without ill effect in mixing portland cement concrete, it should never be used with aluminous cement concretes.

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Pile Driving Vibrations (LR 46-44)

A reader presents an "on-the-job" problem.

In the construction of a highway bridge in a valley which averages about three quarters of a mile wide with the hills rising very rapidly to 1000 ft or more above the valley floor on each side, it was necessary to drive piling to support the abutments and piers. The water table is above 5 ft below natural ground and the foundations for the concrete pile caps are approximately 12 ft below natural ground.

During the driving of piles in the first footing it was noted that a vibration could be felt in the vicinity of the adjacent footing thus bringing up the question of what effect the vibrations caused by the pile driving would have on concrete during and after placement.

The bridge consists of three 80-ft spans, the piles are spaced approximately 3 ft on centers and are covered with a 4 ft thick reinforced concrete cap, on top of which is placed the pier or abutment stems. After placing the concrete cap of one pier, while pile driving was in progress on another footing 160 ft away, a pan of water was set on the top of the cap and protected from air currents to try to determine if any results of vibrations could be noted. It was impossible to distinguish any movement of the water while the piles were being driven, neither was it possible to feel any vibration. Then the question was raised if there was a possibility of damage to concrete being placed while piles were being driven, since it was impossible to distinguish any vibration in the footing.

Corrosion of Reinforcement in Flat Slabs (LR 46-45)

The problem of testing flat slabs with corroded reinforcing is presented.

We have an unusual condition in a large reinforced concrete flat slab building. The slabs supported large brine tanks and pumps. Occasionally this equipment overflowed and leaks were present for some time before they were stopped. This permitted brine to be on the surface of the floor and eventually penetrate into the cracks. We noticed recently some heaving of the slab around the columns and upon investigation discovered that the reinforcing steel at the columns

was rusted out. We are making plans to repair this building, but to foresee the extent of repair we will want to know how extensive this destruction by corrosion of the reinforcement has been.

Is there any way that we can tell how extensive this damage is by safely testing the structure in some manner? Loading of the slabs to any amount approaching its design capacity would be very difficult because of the concentration of equipment in the area.

The problem presented is one which probably does not have a pat answer. An excellent paper by J. Fruchtbaum, "Fire Damage to General Mills Building and Its Repair," in the January 1941, ACI JOURNAL, p. 201, describes the treatment of a parallel and more serious problem.

For discussion of the penetration of corrosion along reinforcing bars, the following references are suggested:

"What Concrete Coverage to Protect Reinforcing," JPP 38-39, ACI JOURNAL, Feb. 1942, *Proc.* V. 38, p. 360.

Tremper, Bailey, "The Corrosion of Reinforcing Steel in Cracked Concrete," ACI JOURNAL, June 1947, *Proc.* V. 43, p. 1137.

Lyse, Inge, "Deterioration of Concrete in Brine Storage Tanks," ACI JOURNAL, Oct. 1947, *Proc.* V. 44, p. 141.

Further comments are invited.

—EDITOR

Discover New-Type Cement, Norwegians Say (LR 46-46)

OIVIND MOE, Sverdrup and Parcel, Inc., St. Louis, Mo., comments on a review carried in the November JOURNAL.

In the November JOURNAL, p. 232, there is a review from *Foreign Commerce Weekly*, titled "Discover new-type cement, Norwegians say."

I would like to call your attention to the fact that the effect on concrete of adding calcite to the cement has been rather heatedly discussed for some time in the technical press (and also the newspapers) of Norway. While some laboratories claim that calicite has a marked favorable effect on concrete, Prof. Inge Lyse at the Norwegian Institute of Technology seems to have arrived at opposite conclusions. In an article in *Teknisk Ukeblad* (Journal of the Norwegian Society of Engineers) for Oct. 27, 1949, Prof. Lyse gives results of tests made during the last year in the concrete laboratory of the institute. These tests also included concrete in which the cement was made of 75 percent portland cement and 25 percent calcite. This cement gave a concrete of consider-

ably lower crushing strength than did straight portland cement. Cubes were tested after 7, 28 and 90 days, and also after 1 year.

Freezing and thawing tests turned out to be equally unfavorable for the calcite concrete. Compared to the straight portland cement concrete, the calcite showed so little resistance to freezing and thawing that Prof. Lyse concludes his article with a warning to be especially careful in this respect when using filler-admixtures in cement and concrete.

Progress, Air Entrainment, Slump and Grading (LR 46-47)

E. COPELAND SNELGROVE, Consulting Engineer, Littleborough, Lancashire, England, discusses new developments in concrete construction.

There is growing evidence that we in England are at length taking note of the lead the United States has given in the use of air entrainment.

In general, except for an area in West Yorkshire, which is afflicted with a harsh quartize gravel, aggregates give no practical trouble in Britain and to this fact may be attributed much of our inattention to new developments.

Chief among the aggregates available are—granite, mountain limestone, olite, flint gravels, various sand stones (and grits), beach pebbles and river gravels (Thames, Severn, Mersey, Trent). The latter are widely distributed and provide the bulk of coarse material for concrete. There is an abundance of good sand. All materials are washed and grading is specified by all leading authorities.

We are, however, much retarded by our system of utilizing available knowledge. Architects who control most of the country's building are concerned with form and color. Not all have studied engineer's structures and many are at the mercy of specialist salesmen.

Also, government departments cannot approve a product until they have made sure. Making sure may take 20 years and a principal scientific official though 15 years behind the times, is able to think himself in the forefront of erudition.

Such is the only feasible explanation for unblushing ignorance of air entrainment and autogenous healing still abounding in high places.

Singleton-Green's two volume book on concrete engineering (published 1933-1935) remains the best work of useful information on concrete originating in England.

Though the grading of aggregates is generally specified, the observation of our Research Stations is that no more than 10 percent of the work so planned enjoys the intended benefit. The small particles do

not obligingly nestle beside their next larger fellows—unless the mix is unusually dry or plasticized. High class specialists adopt elaborate field precautions.

Some few years ago there was a process which used a grout to which a surface active agent was added but only one firm, namely, the Concrete Plasticizing Co. has, so far, set out to develop the art of plasticizing normal concrete by reducing the surface tension of the mixing water.

The number of users, though increasing, is not great. Nevertheless, points of interest have already emerged.

The surface active agent designed specifically for plasticizing concrete is called "Alpennite 105." It is particularly successful—(1) in no-fines concretes, (2) for preventing the harshness and bleeding in any mix, (3) for concrete laid on a steep slope and (4) for making grading effective.

Fig. 1 shows the Cycle Race Track at the Manchester Athletics Club, Fallow Field, under construction.

The slope of the bank is 1 in 1.7. The mix was 4½ cu ft of graded aggregate to one bag (112 lb) of cement. The water-cement ratio was 0.41 with a dose of Alpennite 105 equal to 1/1000 of the total mixing water. A steel float finish was specified.

As illustrated, the apron was laid in alternate bays, each reinforced and with projecting bars to key with the adjacent slabs.

Owing to the very loose nature of the material forming the banks it was necessary to place asbestos sheeting on which to pour the concrete.

The laying of concrete in such places invites reflection on the use of the word slump. The slump test was originally devised as a means of enabling a foreman to adhere to a particular specification of concrete wetness in the field, with one set of ingredients only. It was never intended as a criterion nor to be a means of comparing different mixes and yet, of late, "slump" has been forced into this duty.

Fig. 1—Cycle Race Track, Fallow Field, under construction

When a surface active agent forms part of a mix, the slump test fails completely as in the case at Fallows Field where the "no slump" mix had a steel float workabiliy.

Again a traditional concrete mix having a slump of 1 in. will slump 5 in. when a normal dose of surface active agent is added.

Or, by increasing the air entrainment, as in the case of some lightweight concretes, the mass may be whisked up into a state resembling whipped cream.

We are in fact confronted with three items which deserve further study—(1) the action of entrained or integrated air, (2) the spreading of the mixing water intimately over all particles so that a colloidal state is created and (3) dispersion of fine particles which counteracts segregation.

Slump is a near relative of segregation and the term, restricted as it is in application with mixes of traditional concrete, is inappropriate for plasticized and air-entraining concretes.

Soil Pressure Under a Continuous Foundation Slab (LR 46-26)*

M. HIRSCHTHAL, Delaware, Lackawanna and Western Railroad, Hoboken, N. J., supplies an answer to a question asked in the September JOURNAL.

The same question of soil pressure distribution under continuous column footings, mentioned in the September JOURNAL, was encountered by the writer some 20 years ago. When checking the reactions of the upward pressure, he found that with a uniform width of footing the shears resulting from the soil pressures at the supports did not sum up to the amount of the column reactions, indicating possible instability.

The writer at that time suggested to a western university research professor that this might be an interesting and valuable research problem, but nothing has apparently been done in this line to the writer's knowledge.

In the meantime to allay the fear or uncertainty of this type of design, the footing may be so designed that, instead of having a uniform width of slab, the width is varied to the extent necessary to transmit the soil reactions at the various columns in proportions to the various column reactions, based on a straight line variation of unit soil pressure resulting from the eccentricity of the resultant of the column loads. While the soil pressure distribution most probably is not quite a straight line one, it is safe to assume such distribution in the footing design.

The case is simply an extension of the procedure for the design of a "combined" footing of two columns with unequal reactions, where a trapezoidal footing slab is used to transmit these reactions to the soil.

*See also ACI JOURNAL, Sept. 1949, *Proc.* V. 46, p. 71 and Dec. 1949, *Proc.* V. 46, p. 307.

Construction of Hexagonal Bins (LR 46-28)*

G. L. BLANCHARD, Consulting Engineer, Chicago, Ill., discusses a problem raised in the October JOURNAL.

From rather extensive experience in silo design and construction in northern France and French North Africa (grain elevators), I think I may help in completing the partial answers already given in the October JOURNAL.

I will leave aside the question of cracks in the walls of multicellular cylindrical silos. A complete answer to the problem would require at least a whole chapter in a normal textbook. However, in my opinion, oversimplified methods of design, disregard of local conditions, overlooking the fatigue of the concrete under constant variations of temperature and sun rays ,combined sometimes with hasty pouring of concrete, are the main causes of such accidents.

I agree that under comparable conditions of site and layout, multicellular silos of hexagonal section compare closely to rationally designed circular silos as to the cost per cubic unit of storage capacity.

It is, however, obvious that no general conclusion should be inferred from the study of a particular case.

As to the question of existing elevators of hexagonal section, I may say that this design has been successfully adopted in several instances for elevators of medium storage capacity and a cellular arrangement compacted around a center. A well known French firm of high standing has built, to my knowledge, at least one group of elevators of this type with satisfactory results over a number of years.

J. M. STEINBRUGGE, Moffat and Nichol, Inc., Long Beach, Calif., adds further information on the economy of hexagonal bins.

Our concern considers that the theoretical economy of circular over hexagonal bins can be overcome if precast concrete sections are used.

This was mere conjecture until two years ago, but now that two plants have been built and are in operation, we feel confident of it.

The first plant is six bins wide and nine bins long with a capacity of 660,000 cu ft. The short diameter of the bin is 20 ft, and the 6-in. concrete walls are 35 ft high. This plant was constructed by precasting the wall sections and pouring the corners in place between the wall sections.

The second plant is three bins wide and eleven bins long with a capacity of 360,000 cu ft. The individual bins are about the same dimensions as in the first job. In this case the corners were precast and the wall panels were poured in place.

*See also ACI JOURNAL, Oct. 1949, *Proc.* V. 46, p. 141.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Prismoidal thin-slab structures—I

A. J. Ashdown, *Concrete and Constructional Engineering* (London), V. 44, No. 9, Sept. 1949, pp. 279-285 — Reviewed by Glenn Murphy

In this article, the writer describes methods for calculating stresses in thin-slab hipped roofs of a uniform section. An illustrative example is included.

Joints in the floors and roof of a large factory

Concrete and Constructional Engineering (London), V. 44, No. 9, Sept. 1949, pp. 287-288 — Reviewed by Glenn Murphy

The details of a smooth, dustless joint designed for a floor of a new factory of the British Nylon Spinners, Ltd., at Pontypool, Glamorgan, is described. Detailed drawings are shown.

Sand to six-inch stone—all from same quarry

Engineering News-Record, V. 143, No. 14, Oct. 6, 1949, pp. 30-31 — Reviewed by S. J. Chamberlin

Sand as well as four sizes of crushed rock are obtained from a tight and massive granite-gneiss formation to avoid thermal-expansion differentials among the particles in the hardened concrete for Buggs Island (Va.) Dam. The sand plant, through which materials are water-borne, consists of a rod mill and a bowl classifier. The rod mill is a steel drum containing 27 tons of 12-ft steel rods 2 to 3 in. in diameter.

Prestressed concrete columns at Newport, Monmouthshire

Concrete and Constructional Engineering (London), V. 44, No. 9, Sept. 1949, pp. 289-292 — Reviewed by Glenn Murphy

The extension of a factory of the South Wales Board Mills, Ltd., at Alexandra Docks, Newport, Monmouthshire, involves 104 reinforced concrete columns, one-half of which are solid and the remainder of which are hollow. The solid columns, 12 x 30 in. in cross section, are reinforced with 5 prestressed cables, while the hollow blocks contain 4 prestressed cables. The Freyssinet type of prestressing is employed.

Concrete for decorative purposes (Beton voor representatief binnenwerk)

A. J. Kropholler, *Cement* (Amsterdam), V. 1, No. 5-6, 1949, pp. 82-84 — Reviewed by J. W. T. Van Erp

One of the largest bank buildings in Rotterdam had decorative ceilings in woodwork of great beauty. They were destroyed during the war and in the newly erected struc-

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ture decorative ceilings were installed in which the concrete beams and girders were left unpainted but with a washed gravel finish, a treatment which proved to be very effective for the purpose and yielded results comparing favorably with the old oaken beams.

Concrete aqueduct (Afvoerleing in gewapend beton)

C. W. J. Groothoff, *Cement* (Amsterdam), V. 1, No. 5-6, 1949, pp. 88-93 Reviewed by J. W. T. Van Erp

Sludge from the washing plant of a mine is removed through a long conduit, originally of steel, but now replaced by reinforced concrete. The aqueduct consists of 60-ft lengths separated with good results by expansion joints made of conveyor belting. The cross section of the conduit is U-shaped with removable cover plates. It is supported by I-section columns, about 20 ft high and 33 ft on centers. The structure was designed not only for water and wind pressure but also for longitudinal loads resulting from stoppages in the conduit. The inside is entirely covered by clay tiles.

Prestressed concrete in a large department store (Voorgespannen beton)

G. Baar, *Cement* (Amsterdam), V. 1, No. 5-6, 1949, pp. 94-97 Reviewed by J. W. T. Van Erp

This department store is the first multistory prestressed concrete structure in the Netherlands. Since there are no complicated rigid bents this type of construction allows a considerable reduction in building height. The structure is divided by expansion joints 120 ft apart. Floor construction consists of I-section beams of 20 ft span, 5 ft on centers and girders of 40 ft span, 20 ft on centers. Live loads on floors are 90 lb per sq ft. Prestressing is according to the Blaton-Magnel system, anchoring the steel wires only at the extreme ends of each I-beam.

Gaging wire tension in prestressed tanks

Engineering News-Record, V. 143, No. 16, Oct. 6, 1949, pp. 33-34 Reviewed by S. J. Chamberlin

Two-foot squares of small cold-rolled rods were attached to the tanks and the wire wrappings were drawn in the regular wrapping procedure across these squares or bridges. About 30 wire wrappings fell across each bridge. Plucking was done with an ordinary pencil, with the frequency vibration conducted via a magnetic pickup to an oscillograph for determination of the frequency. A laboratory calibrated "frequency vs tension" chart was then used to evaluate the tension in the wire. It was found that overstressing (the specified stress was 120,000 to 150,000 psi) was as much of a problem as understressing. Selected bridges were left uncovered for future testing.

A factory with thin-slab roofs at Brynmawr, South Wales

Concrete and Constructional Engineering (London), V. 44, No. 9, Sept. 1949, pp. 269-277 Reviewed by Glenn Murphy

The main building under construction is 325 ft wide and about 450 ft long, comprising a production floor flanked on three sides by supplementary rooms. The production floor is supported by 150 eighteen-in. diameter columns, and consists of a flat slab varying in thickness from 7½ to 11 in. The design load is 250 lb per sq ft. The principal feature of the factory is the roof which consists of 9 rectangular domes, 70 ft x 90 ft, with an 8-in. rise. Each dome, which consists of a 3-in. slab, is reinforced with two layers of wire fabirc, supplemented by ½-in. bars at the corners where tensile stresses occur. Each dome is supported around the four edges by girders. Construction details are described.

Base reservoirs of concrete for low-pressure gas holders (Beton voor gashouderkuipen)
J. F. DE WILDT, *Cement* (Amsterdam), V. 1, No. 3-4, 1949, pp. 58-60 Reviewed by J. W. T. VAN ERP

It was found that the total cost of construction of gasholders could be reduced 12.5 percent by making the base reservoir of relatively light concrete instead of steel. The reservoirs of 75 ft internal diameter have a circular expansion joint in the base slab, made watertight by a strip of lead. Movable forms were used, making careful execution essential to maintain the exact interior dimensions demanded by the helical movement of the telescoping upper sections. As the water in the base reservoir gradually becomes more and more chemically aggressive the concrete was protected by a $\frac{1}{8}$-in. coating of asphalt. On top of the water a layer of floating tar-oil soon formed, threatening the dissolution of the asphalt coat. To prevent this a $\frac{5}{32}$-in steel sheet was installed at the proper elevation and backfilled by a $\frac{1}{2}$-in layer of asphalt between the steel and concrete.

Prefabricated units (Betonelementen-bouw)
J. F. DE WILDT, *Cement* (Amsterdam), V. 1, No. 5-6, 1949, pp. 75-81 Reviewed by J. W. T. VAN ERP

This article describes the erection of the gasholders mentioned in the preceding review. Besides the extensive use of prefabricated units, an important item was the use of rigid truss-like reinforcement units in the longitudinal walls. This resulted in a considerable saving in formwork, as it avoided the necessity of extensive timber shoring which is still scarce in the Netherlands. In addition, material handling was reduced. The rigid truss-like reinforcement units were prefabricated and erected by welding them to heavy dowels sticking out of the foundation slab. The prefabricated roof trusses were parabola-shaped three-hinged arches cast in the center of the floor area and erected by means of ginpoles. The roof deckplates were erected from a crane travelling on top of the newly erected trusses.

The whole structure was erected practically without scaffolding, formwork was minimized. Careful execution guaranteed accuracy of dimensions, a prerequisite of speedy erection.

Prestressed concrete bents for overhead catenary wires for electric railroad traction (Draagconstructies voor electrische bovenleiding)
J. THYSSE, *Cement* (Amsterdam), V. 1, No. 3-4, 1949 pp. 64-65 Reviewed by J. W. T. VAN ERP

The scarcity of steel and timber made it imperative to use concrete bents, consisting of trussed posts and an overhead girder of inverted U-section to support the contact wires. The truss-shaped posts were poured as one unit with the web members in tubular formwork. The concrete was poured into the forms of one flange and by vibration gradually filled out the tubular web-members finally extending into the formwork of the other flange, an operation taking about one hour. Directly after pouring, the unit was heated in an oven. Only the reinforcement in the flanges was prestressed. In comparison with equivalent steel construction, prestressed concrete used only $\frac{1}{6}$ as much steel, while the weight of the prestressed bent was 2.75 times that of the steel portals.

Reinforced concrete in mines (Gewapend betonwerken op een diepte van 540 m)
C. W. J. GROOTHOFF, *Cement* (Amsterdam),
V. 1, No. 3-4, 1949, pp. 47-52 Reviewed by J. W. T. VAN ERP

Pumproom, locomotive terminal, storage space and galleries were built at -1800 ft in the Julia mine. Concrete was forced into the formwork by pumping and as the rock bears directly against it voids were minimized. The initial problem in the design was

the exact determination of the load to be expected. Many different theoretical approaches have been made to this problem, the oldest one being that of the "hydrostatic pressure," which gave results far in excess of the pressures observed in actual practice. Best results were obtained by assuming the existence of a strainless body of rock delineated by a parabola the width of the tunnel section and of a height depending on the depth below grade and the characteristics of the rock. Cross sections of the various tunnel-like spaces varied in width, but they were all covered by a segmental arch of the same radius. This proved advantageous as the movable formwork could be standardized and the intersections simplified.

New casting and prestressing techniques for ultra-strong concrete pipe

Engineering News-Record, V. 143, No. 14, Oct. 6, 1949, pp. 24-26 Reviewed by S. J. CHAMBERLIN

Production of 54-in. by 16-ft concrete pipe for a design pressure of 108 psi involves: (1) Roller-compaction of a dry, high-strength concrete in a rotating mold to make a thin-walled pipe, called the core; (2) wirewrapping the core spirally to prestress it simultaneously in two directions; and (3) covering the wire with air-applied mortar. The outer form assembly consists of cast steel end rings, two external semi-cylindrical steel shells, and a pre-tied reinforcing cage. The casting mold hangs loosely from a 16-in diameter smooth steel spindle that projects through the bore of the mold. The mold assembly is rotated at 60 rpm by the revolving spindle. The stiff concrete mixture is conveyed into the form by a cantilevered belt conveyer, flattened smoothly by the centrifugal action and consolidated and compacted by the roller action of the spindle. After steam curing, the core is prestressed in a lathe-like machine in a basket weave design with wire stressed to 140,000 psi. The pitch of the wire determines the magnitude of the longitudinal and circumferential components of the prestress. The prestressed pipe is then left on the lathe for pneumatical application of a ¾-in protective coating of mortar. After final curing, the interior surface is painted with an asphaltic seal coat. Tests indicate that the pipe will withstand loads more than three times as severe as those required for reinforced concrete sewer pipe.

Prestressed concrete (Voorgespannen beton)

A. M. HAAS, *Cement* (Amsterdam), V. 1, No. 3-4, 1949, pp. 61-63 Reviewed by J. W. T. VAN ERP

The present status of prestressing is reviewed and the author concludes that it is still being held back by relative unfamiliarity with it. In prestressed concrete tensile stresses in the concrete are eliminated; the function of the steel is not only to take all tensile stresses in bending, but primarily to impart compression to that part of the concrete which under load would undergo tensile stresses. Experience has shown that for bending moments from 35-700 kip ft a prestressed concrete member of a certain cross section will be able to take moments 2.3 to 3.4 times as large as an ordinary reinforced concrete member of the same cross section. This figure does not indicate however the relative costs; generally speaking prestressed concrete is economical for heavy loads and large spans. It permits the use of extremely slender members with normal loads.

The difficulties experienced with prestressed concrete lie not so much in the design and calculation as in the actual fabrication and erection. Details which in ordinary reinforced concrete are of minor importance now demand careful consideration due to the greater stresses involved. This refers not only to the methods of applying and preserving the prestress in the steel (and the concrete) but also in obtaining concrete of the best possible compressive quality.

The article concludes with a description of an underground garage covered by prestressed girders of 58-ft span.

Rosin and abietic resin as plasticizers for portland cement

Yu. M. Butt and T. M. Berkovich, *Doklady Akad. Nauk SSSR*, V. 66, No. 3, 1949, pp. 443-46

Ceramic Abstracts Nov. 1949

Ordinary, alite, belite, and celite portland cements were ground to a specific surface of 3100 to 3200 sq cm per gm., mixed with 0.1 percent of rosin and abietic resin and 3 percent $CaSO_4 \cdot 0.5H_2O$ by weight of cement, and tested in plastic solutions (1:3). Abietic resin increased the strength of alite and ordinary cements, caused no increase in strength of belite cements, and considerably reduced the strength of cements containing much celite. Rosin had no harmful effect on the strength of alite and ordinary cements, lowered the strength of belite cements, and caused a considerable drop in the strength of cement having much celite. Rosin produced a substantial plasticizing effect in all the cements, but abietic resin affected only the alite and ordinary cements. The effects of rosin and abietic resin on the plasticity and strength of the cements varied with the grain composition of the cements; the plasticizing effect was greater, the more intensive the absorption of the admixtures on the clinker grains, but this, in turn, lowered the rate of hydration of the clinker with a resulting drop in the strength of the hardened cement. Thus, rosin, which had a good plasticizing effect on all the cements, formed relatively dense strength absorption films which had no harmful effect on the strength only in the case of sufficiently active cements with a normal or increased alite content; abietic resin, which had a relatively weaker plasticizing effect on belite cements, formed "liquid" absorptions films which could have had no harmful effect on the hydration of the cements. Rosin and abietic resin in the unsaponified state may be used for increasing the plasticity and life of cements and concretes without reducing the strength; the use of abietic resin for this purpose will not lower the strength of other cements except those with high celite content.

Iron blast-furnace slag—production, processing, properties and uses

U. S. Bureau of Mines *Bulletin* 479. 304 pp. Available from Superintendent of Documents, U. S. Government Printing Office, Washington, D. C. 75 cents.

The first comprehensive summary of information published in the English language on the production, processing and uses of iron blast-furnace slag, the bulletin was published by the Bureau of Mines in cooperation with the National Slag Assn. of Washington, D. C., which supplied technical data and other information.

In the United States, iron blast-furnace slag is processed to produce four general types of commercial slag: screened air-cooled, used chiefly as aggregate and for railroad ballast; unscreened air-cooled, employed principally in base courses for roads and as a fill material; granulated slag, for filling purposes, in highway pavement base courses, and in the manufacture of cement; and lightweight slag, used in molded concrete products.

Most of the commercial slag is used in solid form, but it also serves as a chemical or ceramic raw material in the production of mineral wool and cement.

The history and development of the slag industry as well as the production and processing of iron blast-furnace slag are described in detail in the bulletin, which also discusses the chemical and mineralogical composition of iron blast-furnace slag, its physical properties and specific uses. More than 100 illustrations and a comprehensive bibliography containing more than 550 references are included. Appendixes cover materials required for slag concrete, slag mortar, and asphalt concrete, specifications and methods of testing slag, miscellaneous test procedures and tables.

Single copies of this Bulletin, in special bound buckram covers, are available (so long as the supply lasts) to all those interested in the use of slag, upon request to the National Slag Assn., 644 Warner Building, Washington 4, D. C., or any of its members.

Calcite and other filler-materials in portland cement (Calsitt og andre fillermaterialer i Portland sement)

INGE LYSE, *Teknisk Ukeblad* (Oslo), V. 96, No. 43, Oct. 27, 1949, pp. 813-818

Reviewed by EIVIND HOGNESTAD

The November ACI JOURNAL carried a review entitled "Discover new-type cement, Norwegians say." This reviewer finds it proper to point out that the use of calcite in portland cement concrete has been, and still is, subject to considerable discussion. Some of the claims made by the discoverer of this admixture, Mr. A. Daniels, are opposed by Norway's most outstanding concrete technologist, Prof. Inge Lyse.

In the present article, Professor Lyse refers to an earlier paper on calcite where he concluded that the calcite had the same effect as other finely pulverized stone-materials, namely the so-called filler effect. The present article presents a study of calcite and other filler materials. A mixture of 75 percent portland cement clinker and 25 percent filler material was ground in a laboratory mill to the usual fineness of cement. Seven types of fillers were studied, the first being portland cement clinker itself, thus giving a pure portland cement. The other materials were: the metamorphic limestone calcite, two other types of limestone, two types of shale, and finally quartz. Cubes of four qualities of concrete made with each of these seven types of filler-cement were tested for compressive strength after 7, 28, 90 and 360 days, and subjected to repeated freezing and thawing tests after 7 days of curing.

The compression tests showed that all filler-cements gave lower strength in all qualities and ages than pure portland cement (with 25 percent clinker as "filler"); but the filler-cements gave higher strengths than a cement without filler. Calcite was not superior to the other filler materials in these strength tests.

The freezing and thawing tests showed a substantial reduction in frost resistance for all filler-cements in all concrete qualities. Quartz was somewhat better than the other fillers; but still far inferior to pure portland cement. Professor Lyse concludes that this point is very important when filler-cements are considered for structures exposed to repeated freezing and thawing, and that is most structures exposed to Norwegian weather.

Volume changes observed in small concrete cylinders during freezing and thawing using a mercury displacement dilatometer

RUDOLPH C. VALORE, JR., *Journal of Research*, National Bureau of Standards, V. 43, No. 1, July 1949

HIGHWAY RESEARCH ABSTRACTS Oct. 1949

A mercury-displacement dilatometer for observing volume changes in moist, porous building materials during freezing and thawing is described.

Results are confined to representative volume-temperature relationships for small cylindrical specimens cast from a concrete of one-mix design (plain and modified by the additions of an air-entraining agent) and made with one brand of normal portland cement and Potomac River aggregate.

The moisture condition of the specimens ranged from air-dry to vacuum-saturated. Most tests were performed upon specimens in a state of partial saturation, attained during various curing period. These specimens, when tested, contained from 65 to more than 85 percent of the total water contained after being oven-dried, evacuated, and resaturated following the freezing and thawing tests.

The freezing and thawing cycles employed varied in rates of cooling and heating between 40 to 20 F. These cycles included a slow cycle of 32 hours, and a fast cycle, which took 1 hour for completion.

Volume changes for all specimens tested in the air-dry condition were directly proportional to changes in temperature and independent of the rate of temperature change. The slopes of the volume-temperature relationships were measures of the thermal coefficients of expansion. Volume changes for the partially saturated specimens were generally not independent of the rate of temperature change, and showed departures from a uniform volume-temperature relationship described for the air dry specimens. These departures were believed to be effects of the conversion of water to ice (or ice to water) within the pore structures of the specimens.

The immediate effect of the freezing of water was a distension, which appeared to be directly related to the rate of cooling those cases in which supercooling was minimized or absent. Secondary effects of freezing were shrinkage and relaxation phenomena, which consisted of contractions in addition to the normal thermal contraction of the solid volume of a specimen.

The specimens usually showed a permanent "set" or residual expansion at the completion of a cycle. The residual expansions for repeated cycles were cumulative.

During slow cooling the pore water in a virgin specimen apparently was supercooled before it froze. The amount of supercooling diminished in succeeding repetitions of the slow cycle. Except that when the specimen was maintained unfrozen for a sufficient time after any cycle, supercooling in the following slow cycle was again pronounced. There was no evidence of supercooling during the fast cycle.

Vacuum-saturated specimens subjected to one slow cycle showed distensions upon freezing, and residual expansions at the end of the cycle, more than 30 times as great as the corresponding volume changes in partially saturated specimens similarly tested. According to accepted criteria for soundness, these specimens definitely would have failed during one cycle of freezing and thawing.

Symposium on concrete pavements

Cement och Betong (Malmo), V. 24, No. 2, June 1949, pp. 63-125 Reviewed by EIVIND HOGNESTAD

Swedish roads today (Svensk vagstandard)

B. LILJEQVIST

The present condition of Swedish roads is not in accord with the requirements of modern traffic. The author points out the economic advantages, to the authorities building and maintaining highways as well as to the users of the roads, of an increased paving program.

The economy of concrete pavements in Sweden (Aro betongbelaggningar ekonomiskt berattigade i Sverige)

H. ROHFORS

A more intense, rapid and heavy highway traffic all over the world has led to concrete pavements. The properties and economy of concrete pavements are discussed with the conclusion that such pavements are in a favorable position.

Concrete pavements for roads and airfields (Betongbelaggningar pa vagar och flygfalt)

O. BEIJER

Plain and reinforced concrete pavements are discussed under various subgrade and load conditions. Some practical details regarding the quality of concrete and the pouring methods are presented.

Joints in concrete pavements (Fogar i betongbelaggningar)

S. BERGSTROM

Joints are considered to be necessary from practical as well as economical points of view. The choice of suitable types of joints as well as the details and design of various types are discussed in view of tests at the Cement and Concrete Institute in Stockholm.

The strength of concrete and asphalt pavements (Barigheten hos betongvagar relativt asfaltvagar)
N. ODEMARK

Due to the heavy loads on present main highways, the choice of pavement is generally one between concrete and asphalt. The article reports some investigations regarding the necessary thickness of subgrade and pavements carried out by the Swedish State Bureau of Roads.

Adjustments of concrete pavements by means of the mud jack (Justering av betongbelaggningar medelst Mud-Jack)
F. HELD

The article describes the mud jack as applied to the maintenance of concrete pavements.

Wear of a concrete pavement during 23 years. (Avnotning under 23 ar pa en cementbetongvag)
W. MANNERFELT

Measurements on one of Sweden's oldest concrete pavements of the modern type, show an average of 0.135 mm wear in 23 years of service. Thus a concrete pavement has a remarkable ability to resist wear.

Reflecting curbs (Refflade betongkantstod)
F. SCHIOTT

This article reports an experiment where a reflecting strip 60 cm wide was provided between a concrete highway and the lanes for bicycles on both sides. The details are similar to those for the "singing separators" used by the New Jersey Highway Dept.